Atlantic Canada
Wastewater Systems Guidelines
May 2022

ATLANTIC CANADA
WATER & WASTEWATER ASSOCIATION
American Water Works Association Atlantic Canada Section
Water Environment Federation the water quality people

Purpose & Use of Guidelines

Purpose

The purpose of the *Atlantic Canada Wastewater Systems Guidelines* (2022) *(Guidelines)* is to provide a guide for the development of wastewater projects in Atlantic Canada. These Guidelines are an update of the *Atlantic Canada Wastewater Guidelines Manual for Collection, Treatment, and Disposal* (2006) from the Atlantic Canada Water and Wastewater Association. The update includes revisions to technical requirements and a section on the consideration of climate change for the design of climate resilient infrastructure. The *Atlantic Canada Water Supply Guidelines* (2022) were developed separately and provide a guide for the development of water supply projects in Atlantic Canada.

These Guidelines are intended to be used in the evaluation of sanitary wastewater systems, and for the design and preparation of plans and specifications. The Guidelines suggest minimum requirements for items upon which an evaluation of such plans and specifications may be made by Regulatory Authorities, and establish, as far as is practical, a best practice.

THESE GUIDELINES DO NOT ELIMINATE THE NECESSITY FOR DETAILED DESIGN. ENGINEERS WHO USE THIS GUIDELINE IN PREPARING REPORTS, DESIGN DRAWINGS, AND SPECIFICATIONS MUST RECOGNIZE THAT THE DESIGN ENGINEER RETAINS FULL RESPONSIBILITY FOR THEIR WORK.

Support

Support for the updates to the Guidelines was provided by Natural Resources Canada (NRCan) under the Building Regional Adaptation Capacity and Expertise (BRACE) program, a 5-year (2017-2022), $18.0M initiative under the Adaptation and Climate Resilience pillar of the Pan-Canadian Framework on Clean Growth and Climate Change. The purpose of the program is to increase the ability of communities, organizations, small and medium-sized enterprises, and practitioners to access, use, and apply knowledge and tools on climate change adaptation in their work.

Financial support from NRCan has been matched by in-kind contributions from Project Committee members and peers, starting in late 2018. Committee members include representatives from the provinces of Newfoundland and Labrador, New Brunswick, Nova Scotia, and Prince Edward Island, the City of Charlottetown, Halifax Water, and from the Atlantic Canada Water and Wastewater Association (ACWWA). A representative from the Atlantic First Nations Water Authority joined the Project Committee in late 2020.

The Project Committee included the following representatives:

- Water Supply Guidelines Lead: Wendy Krkosek, Halifax Water, B.A.Sc., Ph.D., P.Eng.
- Wastewater Guidelines Lead: Richard MacEwen, City of Charlottetown, M.Sc., FEC, P.Eng.
- ACWWA Executive Director: Clara Shea.
- New Brunswick Department of Environment and Local Government: Sylvie Morton, M.Sc.E., P.Eng.
- Newfoundland and Labrador Department of Environment and Climate Change: Deneen Spracklin, P.Eng.
- Nova Scotia Environment: Denis Tufts, P.Eng.
- Prince Edward Island Department of Environment, Energy and Climate Action: Morley Foy, P.Eng.
- Atlantic First Nations Water Authority: John Lam, P.Eng.

The guidance and assistance from the above representatives and their peers are acknowledged and greatly appreciated.

Climate Change

Understanding climate change and its impacts on wastewater infrastructure is an important and complex need for utilities in Atlantic Canada. Utilities are anticipated to encounter both challenges and opportunities related to addressing the impacts of ongoing climate change. It is anticipated that impacts from climate change will vary widely across Atlantic Canada due to the size and diversity of the region. There are significant regional economic and demographic differences where every System Owner has their own unique set of priorities and finite resources.

Given the regional differences in Atlantic Canada, there is limited value in presenting detailed site-specific climate change parameters, indices, and adaptation design processes in these Guidelines. Instead, these Guidelines aim to build the capacity of utilities and designers seeking to incorporate climate change information and adaptation strategies within infrastructure planning, design, and operations, using accessible climate science resources and methods which are both reputable and reliable.

The Guidelines will focus on climate change adaptation instead of climate change mitigation. Where possible, the Guidelines will identify opportunities to reduce energy consumption and demand in wastewater operations to limit human-induced greenhouse gas emissions.

The Guidelines serve as an introduction to climate change adaptation for wastewater utilities in Atlantic Canada and will highlight the linkages between changing climate and the planning, design, and operations of infrastructure managed by water and wastewater utilities. A new introductory Climate Change (Chapter 2) aims to deliver an overview for the strategies available to gather climate change information, assess impacts and risks, and to implement effective adaptation planning. Throughout these Guidelines, reference is made to climate change considerations for planning, designing, and operating a Wastewater Treatment Plant (WWTP).

Limitations

Users of these Guidelines are advised that requirements for specific issues such as treatment processes, equipment redundancy, disinfection, and treated effluent are not uniform throughout Atlantic Canada, and that the appropriate regulations, standards, and guidelines should be contacted prior to or during an investigation to discuss specific key requirements.

Approval Process

These Guidelines were prepared for use in the design of infrastructure for wastewater systems in Atlantic Canada. Every effort has been made to ensure that the Guidelines are consistent with current technology and environmental considerations. The approval and permit process outlined in these Guidelines is general in nature and is meant to be an overview only. System Owners are advised to familiarize themselves with the requirements of all legislation and policies dealing with wastewater projects in the province where the work is to be undertaken.

The respective provincial legislation, standards, guidelines, policies, and/or contacts may be accessed as follows:

- New Brunswick Department of Environment and Local Government.
- Newfoundland and Labrador Department of Municipal Affairs and Environment.
- Nova Scotia Department of Environment and Climate Change.
- Prince Edward Island Department of Environment, Energy and Climate Action.

Innovation
These Guidelines are not intended to limit innovation on the part of System Owners. Where the designer can show that alternate approaches can produce the desirable results, such approaches may be considered for approval. Emerging technologies and innovative solutions to technical challenges in the water supply and treatment field are continuously unfolding. Where these solutions adopt forms similar to aspects of existing technologies and guidelines, it is expected that consideration be given to apply the guideline principles where appropriate, such that new technologies build from these Guidelines and are based on sound practice and technical principles.

Definition of Terms
The terms used in these Guidelines reflect generally used definitions in the wastewater industry.

Policy/Position Papers
There are a wide range of issues which must be dealt with in the upgrading of existing wastewater collection and treatment systems, or the design and commissioning of new systems. Not all issues are easily categorized and addressed in these Guidelines. In some cases, technical aspects of the issues are still emerging, while others may require greater discussion regarding the context in which they may be used or dealt with.

The Water Environment Federation (WEF) and the Canadian Water and Wastewater Association (CWWA) have developed policy/position papers that reflect the current state of knowledge, experience, and best practices on a variety of topics. Users of these Guidelines are encouraged to review WEF and CWWA's policy/position papers at the following:
- WEF: http://www.wef.org.
- CWWA: http://www.cwwa.ca.

Reference Material
In developing these Guidelines, material from outside sources was reviewed, and guidelines appropriate for conditions in Atlantic Canada were adopted. In some cases, multiple sources are referenced in the Guidelines.

Conflicts
Conflicting statements may have survived the review process. Should conflicting statements be found, readers are directed to contact the Regulatory Authority for the appropriate jurisdiction for clarification.

Comments
Comments on these Guidelines should be forwarded to contact@acwwa.ca.

Table of Contents

List of Figures

List of Tables

Acronyms

A/O	Anerobic and Oxic
A^2/O	Anaerobic/Anoxic/Oxic
ABC	The Association of Boards of Certification
ACWWA	Atlantic Canada Water and Wastewater Association
ADF	Average Daily Flow

AED	Automated External Defibrillator
ANSI	American National Standards Institute
AOR	Actual Oxygen Rate
ASTM	American Society for Testing and Materials
AWS	American Welding Society
BAF	Biological Aerated Filters
BOD	Biochemical Oxygen Demand
BOD_5	5-day Biochemical Oxygen Demand
BRACE	Building Regional Adaptation Capacity and Expertise
CAN-EWLAT	Canadian Extreme Water Level Adaptation Tool
$CBOD_5$	Carbonaceous 5-day Biochemical Oxygen Demand
CCME	Canadian Council of Ministers of the Environment
CCTV	Closed-Circuit Television
CDS	Continuous Deflection Separator
CEQG	Canadian Environmental Quality Guidelines
CFIA	Canadian Food Inspection Agency
CFID	Continuous Feed and Intermittent Discharge
CGA	Canadian Gas Association
CIPP	Cured-in-Place Pipe
CMAS	Complete-Mix Activated Sludge
COD	Chemical Oxygen Demand
CP	Contingency Plan
CSA	Canadian Standards Association
CSO	Combined Sewer Overflows
CSS	Combined Sewer Systems
CT	Contact Time
CWWA	Canadian Water and Wastewater Association
DAF	Dissolved Air Flotation
DCS	Distributed Control Systems
DI	Ductile Iron
DO	Dissolved Oxygen
DPD	Diethyl-p-phenylene diamine
EC	Electrical Conductivity
EQ	Exceptional Quality
ERA	Environmental Risk Assessment
ERP	Emergency Response Plan
F/M	Food-to-Microorganism Ratio
FRP	Fiber Reinforced Plastic
FWS	Free Water Surface
GBT	Gravity Belt Thickeners
GCM	Global Climate Models
GHG	Greenhouse Gas
GP	Grinder Pump
HDPE	High-Density Polyethylene
HLR	Hydraulic Loading Rate
HMI	Human Machine Interface
HOA	Hand-Off-Auto
HVAC	Heating, Ventilation, and Air Conditioning
I&I	Infiltration and Inflow
ICLR	Institute for Catastrophic Loss Reduction
IDF	Intensity-Duration-Frequency

IES	Illuminating Engineering Society
IFAS	Integrated Fixed-Film Activated Sludge
IFID	Intermittent Feed and Intermittent Discharge
IPCC	Intergovernmental Panel on Climate Change
ISO	International Organization for Standardization
LAP	Land Application Plan
LOR	Loss of Rotation
MBAS	Methylene Blue Active Substance
MBBR	Moving Bed Biofilm Reactor
MCC	Motor Control Center
MCRT	Mean Cell Residence Time
MLSS	Mixed Liquor Suspended Solids
MLVSS	Mixed Liquor Volatile Suspended Solids
MPN	Most Probable Number
NFPA	National Fire Protection Association
NMP	Nutrient Management Plan
NOD	Nitrogen Oxygen Demand
NPV	Net-Present-Value
NRCan	Natural Resources Canada
O&M	Operation and Maintenance
OUR	Oxygen Uptake Rate
P&ID	Process (or Piping) and Instrumentation Diagram
PCB	Polychlorinated Biphyenyls
PE	Population Equivalent
PFRP	Processes to Further Reduce Pathogens
PLC	Programmable Logic Controller
PP	Polypropylene Copolymer
PVC	Polyvinyl Chloride
RAS	Return Activated Sludge
RBC	Rotating Biological Contactor
RCM	Regional Climate Models
RCP	Representative Concentration Pathways
RDT	Rotary Drum Thickeners
RFI	Radio Frequency Interference
RI	Rapid Infiltration
RSF	Recirculating Sand Filters
RTF	Recirculating Textile Filter
SAR	Sodium Adsorption Ratio
SBR	Sequencing Batch Reactor
SCADA	Supervisory Control and Data Acquisition
SCC	Standards Council of Canada
SDGS	Small Diameter Gravity Sewers
SDPS	Small Diameter Pressure Sewers
SDS	Safety Data Sheets
SDSM	Statistical DownScaling Mode
SF	Subsurface Flow
SIPP	Sprayed in Place Pipe
SOR	Standard Oxygen Rate
SOUR	Specific Oxygen Uptake Rate
SPEI	Standard Precipitation Evapotranspiration Index
SRT	Solids Retention Time

SS	Suspended Solids
SSO	Sanitary Sewer Overflows
SSP	Shared Socioeconomic Pathways
STEGS	Septic Tank Effluent Gravity Systems
STEPS	Septic Tank Effluent Pump Systems
SVI	Sludge Volume Indices
SVR	Sludge Volume Ratio
$TBOD_5$	Total 5-day Biochemical Oxygen Demand
TDH	Total Dynamic Head
TDS	Total Dissolved Solids
TKN	Total Kjeldahl Nitrogen
TN	Total Nitrogen
TRC	Total Residual Chlorine
TS	Total Solids
TSS	Total Suspended Solids
UPS	Uninterruptable Power Supply
USEPA	United States Environmental Protection Agency
UV	Ultraviolet
UVT	Ultraviolet Transmittance
VFD	Variable-Frequency Drive
VSS	Volatile Suspended Solids
WAS	Waste Activated Sludge
WEF	Water Environment Federation
WHMIS	Workplace Hazardous Materials Information System
WSER	Wastewater Systems Effluent Regulations
WTP	Water Treatment Plant
WWTP	Wastewater Treatment Plant

Appendices

Table of Contents

Chapter 1 Approval Requirements & Procedures

1.1 General Overview

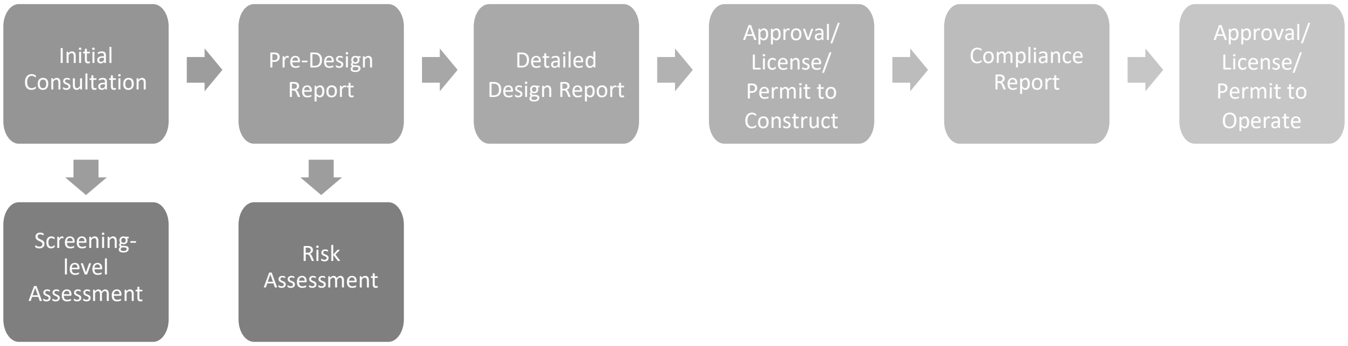
The approval process for a wastewater collection and/or wastewater treatment/effluent discharge system varies from province to province and can include multiple overlapping agencies. In all cases, the System Owner should seek input from the respective province early in the process as to whether an approval for construction, modification, or operation will apply to the project before it advances. In some cases, an approval is needed for the vertical assets (e.g., WWTP) and associated discharge, and pump station but is not required for linear assets, such as extensions to local gravity wastewater collection piping. The application for approval should be submitted, reviewed, and approved prior to starting construction. The application should be signed by the System Owner, in the format prescribed by the respective province.

Depending on the type of infrastructure to be constructed, there may be additional assessments, approvals, permits and/or authorizations from other federal and/or provincial authorities required before proceeding with construction. Determination of an overall regulatory roadmap of a given project is beyond the scope of these Guidelines. These Guidelines are intended to guide the design stages associated with wastewater collection and wastewater treatment/effluent discharge infrastructure. In general, pre-consultation with the Regulatory Authorities is recommended in the project planning stages.

Climate change is creating evolving risks for wastewater infrastructure and operations. Governments and Regulatory Authorities are increasingly requiring consideration of the impacts of climate change on built infrastructure in the planning stage (e.g., as a requirement for funding). These Guidelines provide examples of climate change considerations and potential impacts to support identification of risks and incorporation of adaptation measures into WWTP projects.

1.1.1 Summary of Approval Process

The approval process can be a multi-step procedure and varies from province to province. It is recommended that pre-consultation be conducted with all involved parties including the System Owner, design team, Regulatory Authority, and other key stakeholders. Throughout the project, regular consultation and status review meetings should be continued through the concept, design, approval, and construction stages. The general approach for addressing a project regulatory process is shown on the following chart and expanded upon in the following sections. It is recognized, however, that not all steps are required in each project, or in each province.



Within these Guidelines, climate change is addressed in two phases of the approval process. First, with a screening-level assessment at the pre-consultation phase, and second (if applicable), with a risk assessment at the pre-design phase (refer to Chapter 2 for further guidance). The purpose of the screening-level assessment is to consider potential impacts of climate change on the project, determine the System Owner's priorities regarding risk tolerance, and determine the need for a climate change risk assessment in the pre-design phase. If potential climate change impacts on the project are identified but cannot be directly addressed due to data gaps (such as a need for climate projections and a more thorough design review), then a risk assessment is recommended to be completed during the pre-design phase.

Depending on the nature of the project, the scope of submission to the Regulatory Authority for each phase of approvals will vary in depth and complexity.

In some provinces, the Pre-design Report will be submitted to the Regulatory Authority, with a request for comments. Acceptance of a Pre-design Report (in any form) from a Regulatory Authority should not be considered as having received official approval to proceed with construction or modification of a project.

Where applicable, a processing fee form should be completed, and the appropriate fee submitted.

The formal approval application, with the plans, specifications, and supporting documentation should be submitted at least 90 days (or as specified by the Regulatory Authority) prior to the planned start of the construction or modification project. The plans, specifications, and supporting documentation should be stamped by a Professional Engineer that is licensed in the province where the project is located. The application should be submitted to the Regulatory Authority and should be signed by the System Owner, or where authorization is provided, a person representing the System Owner.

The Regulatory Authority should review the application to determine if it conforms with policies, standards, or guidelines enforced by the department. During the review of the application, the Regulatory Authority may request additional information on the project. If the requested information is not received, the Regulatory Authority may declare the application incomplete and advise the applicant of such.

In some provinces, an Approval/License/Permit to Construct should be issued after the design application has been reviewed and found to be satisfactory for all requirements, including climate resilience. The proposed works should not be undertaken by the System Owner until the official approval has been issued by the Regulatory Authority. The System Owner should be aware of the expiration date associated with the approval. If the project is delayed potentially beyond the expiration date on the approval, an extension will need to be issued by the Regulatory Authority.

In some provinces, a "Post-Construction Report/Certificate of Compliance" is required at the completion of the project.

After the submission of the Post-Construction Report/Certificate of Compliance, the Regulatory Authority may provide an approval if all aspects of the project are acceptable.

The purpose of the permit is to clearly outline the operating and reporting requirements for the wastewater system.

The expiry date of the approval and the terms for renewal would be indicated by the Regulatory Authority.

1.2 Pre-Consultation

System Owners planning a wastewater project should consult with the Regulatory Authority to discuss the scope of the project and to determine the regulatory requirements. Key issues for discussion, may include, but are not limited to, the following:

- Identification of applicable laws, standards, and regulations that apply to the project.
- Identification of applicable permits, approvals, and authorizations required for the project.
- Identification of effluent discharge requirements:
  - Wastewater Systems Effluent Regulations (WSER).
  - Provincial requirements.
- Identification of flow gauging and wastewater characterization studies.
- Identification of Infiltration and Inflow (I&I) investigations.
- Identification of at-source control.
- Identification of assessments, including those which address considerations for climate change. The respective Regulatory Authority should be contacted to determine specific requirements.
- Identification of key timelines associated with the review process.
- Identification of key stakeholders that should be involved in the process.
- Completion of a screening-level assessment to consider potential impacts of climate change on the project (refer to Chapter 2 for further guidance).

A screening-level assessment provides an opportunity to:

- Determine if the project requires an assessment and/or registration under the respective Environmental Assessment Act and additional regulatory requirements regarding climate change.
- Consider potential impacts of climate change on the project.
- Assess the System Owner’s risk tolerance regarding climate change vulnerabilities and risk.
- Determine the need and/or scope of a risk assessment to be completed in the pre-design phase.

1.3 Pre-design Report

A Pre-design Report should be considered as good engineering practice even when not required by the Regulatory Authority.

A pre-design evaluation will generally be required by the Regulatory Authority for large scale projects and/or projects involving the development or upgrade of the following:

- Large gravity sewers.
- Pump stations and force mains.
- Wastewater treatment/treated effluent discharge locations.

The Pre-design Report should document the “problem statement” or the “problem to be solved”, which may or may not be the same as the long-term goals.

The purpose of a Pre-design Report is to assess the existing infrastructure and operating conditions, identify system or asset vulnerability and risk, including potential impacts of climate change, and to determine options for upgrade, improvement, or replacement.

Architectural, structural, mechanical, and electrical conceptual designs are typically not included in the pre-design, evaluation; however, their estimated costs must be evaluated in terms of their impact on the overall project costs. Sketches may be included to describe treatment processes where applicable. Outline specifications of process units and special equipment may also be included.

A pre-design evaluation for a proposed project is typically used by:

- The municipality, System Owner, private developers, or industry, for a project description, including risk assessment, findings, conclusions, cost estimates, financing requirements and recommendations.
- Designers, to establish the overall scope of design, including climate change considerations, and for the arrangement, capacity, and type of components to be designed.
- The Regulatory Authority for evaluation of environmental impacts, examination of process operations, verifying compliance with effluent discharge requirements, and for concept approval, should identify other regulatory triggers (e.g., those which require the inclusion of climate change considerations). Investment groups and government funding agencies to evaluate the "quality" of the proposed project with reference to authorization and financing.
- News media for description of the project.

General practice is that a Pre-design Report may be considered valid for a period of up to 5 years unless new information has resulted in it being obsolete. This practice, however, may vary from project to project or within a given jurisdiction.

1.3.1 Contents of a Pre-Design Report

The pre-design evaluation should, where applicable, include but not be limited to consideration of the following:

- Conduct a risk assessment to evaluate system/asset vulnerability and risk to climate change and identify/prioritize adaptation measures.
- Develop predicted service population.
- Establish a specific service area for immediate consideration and indicate possible extensions.
- Present reliable measurements of flow and analyses of wastewater constituents as a basis of process design.
- Identify existing and potential receiving water uses.
- Identify potential wastewater treatment/effluent discharge sites.
- Estimate of probable costs for immediately proposed facilities.
- Present a reasonable method of financing and show typical financial commitments.
- Suggest an organization and administrative procedure.
- Consider operational requirements with regards to protection of receiving water quality and projected future plant discharge water quality requirements; reflect local bylaws and federal/provincial regulations.
- Include a site plan indicating location of residences, private and public water supplies, recreational areas, watercourses, zoning, floodplains including climate change considerations, and other areas of concern when siting wastewater collection and treatment plants.
- Identify existing problems including Combined Sewer Overflows (CSOs) and Sanitary Sewer Overflows (SSOs) and proposed remedial measures to correct any of the problems.
- Maintenance and operation requirements.
- Identify adaptation/resiliency measures/options.
- Plans for monitoring of adaptation/resiliency measures.
- Any other requirements of the Regulatory Authority.
- Present summarized findings, conclusions, and recommendations for the System Owner's guidance.

1.3.2 Concept & Guidance for Plans & Specifications

The Pre-design Report should be complete so that detailed plans and specifications may be developed from it without substantial alteration of concept and basic considerations. In short, basic thinking, fundamentals and decisions are spelled out in the Pre-design Report and carried out in the detailed design plans and specifications.

1.3.3 General Format of Pre-design Report

The requirements for a Pre-design Report will vary with the scope of the project and the specific requirements of the client and/or Regulatory Authority. Projects involving the consideration of new wastewater collection infrastructure and the location of a new WWTP and new outfall will typically provide detailed requirements for a Pre-design Report in a request for proposals. In the absence of detailed requirements for a Pre-design Report, the following should be considered, as applicable to the project, as a format for the Pre-design Report:

- Project background.
- Screening-level assessment and risk assessment (if applicable) addressing the impacts of climate change.
- Evaluation of wastewater collection options.
- Evaluation of wastewater treatment and outfall location options.
- Discussion of wastewater collection, treatment, and outfall location options.
- Discussion of probable capital/operations and maintenance costs of evaluated options.
- Recommended wastewater collection and treatment process/siting.

Table 1.1 provides a checklist of components that typically should be included in a Pre-design Report. The requirements can be determined by the System Owner by indicating which components are applicable to the project and serve to determine the contents for a specific project to be submitted to the Regulatory Authority for review and approval. Specific tasks to be completed under each component are provided in Appendix A.

Table 1.1: Pre-design Report Requirements Checklist

	Yes / No / N/A
• Introduction.	
– Purpose. – Scope.	
• Background.	
– General. – Economics. – Regulations. – Hydraulic capacity.	
• Existing facilities evaluation.	
– Collection system. – Treatment system. – Process. – Wastewater characteristics.	
• Screening-level assessment and/or risk assessment.	
• Proposed project.	
– Collection system. – Site requirements. – Wastewater characteristics. – Receiving water considerations and assimilative capacity. – Including climate change considerations.	
• Adaptation measures/strategies to be incorporated into design.	
• Alternatives.	
– Alternate process and site. – Selected process and site.	
• Project financing.	
• Legal and other considerations.	
• Appendices.	
• Technical information.	
• Design criteria.	
– Collection system. – Process facilities. – Process diagrams. – Space for personnel, laboratories, and records. – Chemical control. – Support data.	
• Supplemental information (treated effluent to land).	
– Location. – Geology. – Hydrology. – Soils. – Agricultural practice. – Adjacent land use.	

1.4 Detailed Design

The System Owner must prepare and submit an application and, where applicable, a Detailed Design Report to the Regulatory Authority for approval. The application should be signed by the System Owner, or where authorization is provided, by a person representing the System Owner.

Detailed design should include climate-resilient components as identified and prioritized through the risk assessment process completed in the pre-design phase of the project. The climate risks and the climate resiliency strategies incorporated into design should be clearly stated in the Detailed Design Report and on plans/drawings, etc. (where applicable). Clear reasoning should be provided for instances where risks have been identified and resiliency strategies were not incorporated into design (this may be presented within the risk assessment). Clear reasoning should be provided in the case that the conclusions of the screening-level assessment negated the completion of a risk assessment and climate resilient strategies were not incorporated into design (this may be presented within the screening-level assessment). Refer to Chapter 2 for further guidance.

Applications for specific items within the project, such as stream crossings may require approval from other jurisdictions.

An Approval/License/Permit to Construct cannot be issued until final, complete, detailed plans and specifications have been submitted to the Regulatory Authority, reviewed, and found to be satisfactory.

Detailed design documents submitted with the application should include, but not be limited to:

- Design Report.
- Design plans.
- Specifications.
- Quantities and cost estimates.
- Other information as required by the Regulatory Authority.

1.4.1 General Format of a Detailed Design Report

The Detailed Design Report should contain all the up-to-date technical information on the project. It should make use of the data presented in the Pre-design Report but should be a stand-alone document that does not require the Regulatory Authority to refer to the Pre-design Report. Where multiple options may have been considered in the Pre-design Report, only the selected option is to be included in the Detailed Design Report.

The risk assessment, including all pertinent information (e.g., climate projections, data sources, evaluation methodology, results, and identification/prioritization of adaptations strategies), should be included in the Detailed Design Report. The climate resiliency strategies incorporated into design should be clearly stated in the Detailed Design Report. Refer to Chapter 2 for further guidance.

Similar to the Pre-design Report requirements, the requirements for a Detailed Design Report will vary with the scope of the project and the specific requirements of the System Owner and Regulatory Authority. When it is determined that specific project requires a Design Report, Table 1.2 provides a checklist of components that typically should be included in the Report. The requirements can be determined by the System Owner by indicating which components are applicable to the project and serve to determine the contents for a specific project to be submitted to the Regulatory Authority for review and approval. Specific tasks to be completed under each component are provided in Appendix A.

Table 1.2: Detailed Design Report Requirements Checklist

	Yes / No / N/A
• Screening-level assessment and/or risk assessment.	
• Climate resiliency strategies incorporated into design.	
• Collection system.	
• Process facilities.	
• Process diagrams.	
• Laboratory.	
• Operation and Maintenance (O&M).	
• Office space for administrative personnel and records.	
• Personnel services (locker room and lunchroom).	
• Chemical control.	
• Collection system control.	
• Control summary.	
• Support data.	

1.4.2 Plans

1.4.2.1 General

All plans should bear a title showing the project name, location, System Owner, the scale in appropriate units, the north point, date, and the engineer's name and be stamped by a Professional Engineer licensed to practice in the province where the project is located.

The plans should be clear and legible. They should be drawn to scale which will permit all necessary information to be plainly shown. Datum used should be indicated. Locations and logs of test borings, when made, should be shown on the plans.

Detail plans should consist of plan views, elevations, sections, and supplementary views which, together with the specifications and general layouts, provide the working information for the contract and construction of the works. The plans should also include dimensions and geodetic elevations of structures, the location and outline form of equipment, location and size of piping, water levels and ground elevations. The plans should identify existing and future risks to climate change, where applicable, as identified through the risk assessment.

1.4.2.2 Sewers

A comprehensive plan of the existing and proposed sewers should be submitted for projects involving new sewer systems or substantial additions to existing systems. This plan should indicate the following:

- Geographical features.
- Topography and elevations: existing or proposed streets and all streams or water surfaces should be clearly shown. Contour lines at suitable intervals should be included.
- Streams: the direction of flow in all streams and high and low water elevations of all water surfaces at sewer outlets and overflows.

- Boundaries: the boundary lines of the municipality, the sewer district or area to be serviced by sewers, and regulatory setbacks from natural features with distances.
- Sewers.
- Sensitive areas and potential environmental issues.

The plan should show the location, size, and direction of flow of all existing and proposed sanitary and combined sewers draining to the treatment works concerned.

Profiles should have a horizontal scale of not more than 1-in-500 and a vertical scale of not more than 1-in-50. Plans and profiles should indicate:
- Location of streets and sewers.
- Line of ground surface, size, material, and type of pipe, length between manholes, invert and surface elevation at each manhole and grade of sewer between each two adjacent manholes. All manholes should be numbered on the plan and correspondingly numbered on the profile.

Where there is any question of the sewer being sufficiently deep to serve any residence, the elevation and location of the basement floor should be plotted on the profile of the sewer which is to serve the house in question. The engineer should state that all sewers are sufficiently deep to serve adjacent basements except where otherwise noted on the plans, including:
- Locations of all special features such as inverted siphons, concrete encasement, elevated sewers, etc.
- All known existing structures both above and below ground which might interfere with the proposed construction, particularly water mains, gas mains, storm drains, etc.
- Special detail drawings, made to a scale to clearly show the nature of the design, should be furnished to show the following particulars:
  - All stream crossings and sewer outlets, with elevations of the stream bed and of normal and extreme high and low water levels.
  - Details of all special sewer joints and cross-sections and details of all sewer appurtenances such as manholes, lamp holes, inspection chambers, inverted siphons, regulators, tide gates, and elevated sewers.
- Details and plans of CSOs and treatment components according to the Regulatory Authority having jurisdiction.

1.4.2.3 Wastewater Pumping Stations
A plan should be submitted for projects involving construction or revision of pumping stations. This plan should show the following:
- The location and extent of the tributary area.
- Any municipal boundaries with the tributary area.
- The location of the pumping station and force main and pertinent elevations.
- Identify sensitive areas and potential environment issues.
- Identify all required setbacks from natural features with distances.

Detail plans should be submitted showing the following, where applicable:
- Topography of the site.
- Existing pumping station, proposed pumping station, including provisions for installation of future pumps.
- Elevation of high water at the site and maximum elevation of wastewater in the collection system upon occasion of power failure.

- Maximum hydraulic gradient in downstream gravity sewers when all installed pumps are in operation, and test borings and groundwater elevations.
- Details and plans of CSOs and treatment components according to the Regulatory Authority having jurisdiction.
- Potential climate change risks.

1.4.2.4 Wastewater Treatment Plant

A plan should be submitted, showing the WWTP in relation to the remainder of the system.

Sufficient topographic features should be included to indicate its location with relation to streams and the point of discharge of treated effluent.

Identify sensitive areas, potential environmental issues, and potential climate change risks. Table 1.1 provides a checklist of components that typically should be included in a wastewater collection and treatment Pre-design Report. The requirements can be determined by the System Owner by indicating which components are applicable to the project and serve to determine the contents for a specific project to be submitted to the Regulatory Authority for review and approval. Specific tasks to be completed under each component are provided in Appendix A.

Layouts of the proposed WWTP should be submitted, showing:

- Topography of the site.
- Size and location of plant structures.
- Schematic flow diagram showing the flow through various plant units.
- Piping, including any arrangements for by-passing individual units. Materials handled and direction of flow through pipes should be shown.
- Hydraulic profiles showing the flow of wastewater, supernatant, mixed liquor, and sludge.
- Test borings and groundwater elevations.

Detail plans should be submitted showing the following, where applicable:

- Location, dimensions, and elevations of all existing and proposed plant facilities.
- Elevations of high and low water level of the body of water to which the plant effluent is to be discharged.
- Type, size, pertinent features, and manufacturer's rated capacity of all pumps, blowers, motors, and other mechanical devices.
- Minimum, average, and maximum hydraulic flow in profile.
- Adequate description of any features not otherwise covered by specifications or Design Report.
- Potential climate change risks.

1.4.3 Technical Specifications

Complete technical specifications for the construction of sewers, pumping stations, WWTPs and all appurtenances, should accompany the plans.

The specifications accompanying construction drawings should include, but not be limited to:

- All construction information not shown on the drawings which is necessary to inform the builder in detail of the design requirements as to the quality of materials and workmanship and fabrication of the project.
- Type, size, strength, operating characteristics, and rating of equipment.

- Allowable infiltration.
- The complete requirements for all mechanical and electrical equipment, including machinery, valves, piping and jointing of pipe, electrical apparatus, wiring, and meters.
- Laboratory fixtures and equipment.
- Operating tools.
- Construction materials.
- Special filter materials such as stone, sand, gravel, or slag.
- Miscellaneous appurtenances, chemicals when used.
- Instructions for testing materials and equipment as necessary to meet design standards.
- Operating tests for the completed works and component units (it is suggested that these performance tests be conducted at design load conditions wherever practical).

1.4.4 Review of Design Submissions

The application and all supporting documentation should be submitted to the Regulatory Authority. Where applicable, a processing fee form should be completed, and the appropriate fee submitted.

The formal approval application, with the plans, specifications, and supporting documentation, must be submitted prior to the planned start of the construction or modification project, and provide for the requisite number of days allowed for review by respective provincial agency. The plans, specifications and supporting documentation should be stamped by a Professional Engineer that is licensed to practice in the province where the project is located. The application should be signed by the System Owner, or where authorization is provided, a person representing the System Owner.

During the review of the application, the Regulatory Authority may request oral or additional written information on the project. If requested information is not received, the Regulatory Authority may declare the application incomplete, and advise the applicant of such.

1.5 Approval/License/Permit to Construct

An Approval/License/Permit to Construct should be issued by the Regulatory Authority after the design application has been reviewed and found to be satisfactory. The proposed works should not be undertaken by the System Owner or contractor until the official approval has been issued.

The approval will provide the System Owner with the authority to proceed with construction of the project.

Any changes in the approved works or works other than those specified in the application, must be submitted in writing to the Regulatory Authority, and approved, in the form of an amendment to the approval prior to construction.

1.6 Post Construction Report/Certificate of Compliance

A “Post-Construction Report/Certificate of Compliance”, although not required in all jurisdictions, should be provided at the completion of the project. This report may be called a Commissioning Report or Certificate of Compliance, depending on the jurisdiction.

The report should contain all information regarding major changes, if any, from the approved plans or specifications made during construction. These major changes include any deviations which affect capacity, flow, or operation of units. The report should also include results of all test runs of the WWTP to demonstrate that the plant can produce water meeting all applicable standards.

Information required includes, but is not limited to, the following:

- Equipment start-up tests and any other tests results produced during construction.
- Results of start-up of the plant confirming that treated water meets the wastewater effluent quality requirements.
- Confirmation that the plant and its components have been properly disinfected prior to being placed in use.
- Confirmation that a cross-connection survey has been performed and cross connections have not been found.
- Confirmation that all components and chemicals are NSF International compliant.
- Indication that as-built drawings, O&M manuals, and any other relevant documentation have been provided to the System Owner/operator and/or other body if required by the Regulatory Authority.
- Confirmation that operator certification is consistent with provincial requirements.

If specific information cannot be confirmed when the report is submitted, a plan outlining the time frame to comply should be submitted.

1.7 Approval/License/Permit to Operate

When applicable, the Regulatory Authority may provide an Approval/License/Permit to Operate or equivalent document.

The purpose of the approval is to clearly outline the operating and reporting requirements for the wastewater system.

The expiry date of the approval and the terms for renewal would be indicated by the Regulatory Authority.

1.8 Monitoring & Recording Requirements

Monitoring and recording requirements by the System Owner would be outlined, as applicable, by the Regulatory Authority in the approval.

The monitoring program should be carried out in compliance with sampling and analysis requirements outlined in the approval.

Any monitoring carried out by a Regulatory Authority does not relieve the System Owner of their responsibility related to this function. The operator would be employed by the System Owner, therefore, the ultimate responsibility for compliance is from the System Owner.

1.9 Reporting Requirements

Reporting should be carried out in compliance with the requirements outlined in the Regulatory Approval documents or legislation as applicable.

1.10 Facility Classification & Operator Certification

Some provinces have adopted regulations that make facility classification and operator certification mandatory, while others strongly recommend operator certification. Where applicable, the regulations may require all wastewater treatment personnel to be certified and require that an operator with a certification level equivalent or greater to the facility classification be in charge.

The Regulatory Authority should be consulted regarding specific requirements.

1.11 System Owner/Operator Responsibility

The System Owner of any wastewater treatment or collection facility should practice due diligence and should ensure that all monitoring and reporting is conducted in accordance with the requirements of the approval.

In provinces that do not have mandatory facility classification and operator certification, System Owners are encouraged to voluntarily ensure that operators have attained the required certification status and are provided with on-going education and training.

1.12 Regulatory Authority Responsibility

The responsibilities of the Regulatory Authority should be as outlined in the latest respective provincial acts and other applicable regulations, policies, guidelines, and directives.

Table of Contents

Chapter 2 Incorporating Climate Change in the Design of Wastewater Infrastructure

2.1 Introduction

In general, across Atlantic Canada, temperature, and precipitation are increasing, sea-level is rising, extreme weather events are becoming more common, and these changes are projected to continue. Climate change is expected to have direct and indirect impacts on the design and operation of wastewater systems. Projected climate changes and associated impacts will vary across the Atlantic Region due to geographic, economic, and demographic variability. System Owners are anticipated to encounter both challenges and opportunities with climate change. These challenges are often in the form of evolving vulnerabilities and risks to infrastructure and operations, whereas opportunities may be in the form of operational cost savings from increased system efficiency and reduced energy demands. Adaptation of existing and future wastewater infrastructure will minimize the social, environmental, and economic costs associated with severe weather and longer-term climate change trends.

Historically, infrastructure and discharge standards, best practices, and codes that incorporate climate-based risk considerations were established (either whole or in part) on previously measured values and the assumption of a relatively stationary climate. This stationary assumption is challenged by projected climate changes, which may translate to an increased risk of infrastructure failure, presentation of new hazards, and treatment challenges for wastewater System Owners. Proactive investments in resilient design and planning strategies (i.e., one that considers a full range of climate projections and risks), can help to mitigate increasing maintenance requirements, repair and/or replacement costs. For that reason, it is essential that best practices for built infrastructure, treatment, and operations consider both current and future climatic conditions to mitigate loss and ensure the safety of Canadians.

As introduced in Chapter 1, these Guidelines recommend that climate change be incorporated into two phases of the approval process:

1. First, at the pre-consultation phase with a screening-level assessment.
2. Second, at the pre-design phase with a risk assessment (scope of which is determined through the screening-level assessment).

2.2 Purpose

As described in Section 2.1, climate change is relevant not only for proposed systems (i.e., design, approvals, construction), but also for existing systems (i.e., operations, maintenance, retrofitting). For that reason, the purpose of this Chapter is to support teams endeavoring to develop risk assessments in support of adaptation planning for both existing and proposed wastewater infrastructure and operations/maintenance considerations. The information is presented at a level of detail that is appropriate for System Owners (e.g., managers, engineers, and operators) and Design Engineers. Teams involved in these assessments should have both asset and climate science expertise.

This Chapter is not prescriptive. Instead, it provides background concepts, illustrates complexities, and suggests factors to consider when choosing climate information and risk assessment methodologies. There is no list of climate interactions or indices provided, nor is there a step-by-step “how to” section on developing a risk assessment and adaptation plan, since these depend on the goals and scope of the assessment, geography, stakeholders, and availability of climate information. Instead, the purpose of these Guidelines is to illustrate the steps and relevant questions, so that teams can tailor their process to the site-specific project needs. Similarly, instead of providing a go-to list of data portals, climate information is categorized into broader types that reflect the practical realities of where information is sourced, and examples are provided. Thus, the information in this Chapter is applicable for a range of assessment types, impacts, jurisdictions, and climates across the Atlantic Canadian region.

Lastly, this Chapter does not address climate change “mitigation” - which refers to the process of reducing Greenhouse Gas (GHG) emissions through action planning. As introduced above, this Chapter focuses on adaptation (building resilience to the impacts of climate change). It is, however, a best practice to consider whether climate adaptation plans can prioritize actions that also reduce life cycle GHG emissions.

2.3 CSA Standard for Climate Change Adaptation for Wastewater Treatment Plants

CSA S900.1.18 Climate change adaptation for wastewater treatment plants from the Canadian Standards Association (CSA) provides System Owners with a resource for design, operation, and retrofit criteria for the implementation of climate resiliency for WWTPs. The standard outlines a general eight step approach for conducting a risk assessment to address adaptation considerations for WWTPs to a changing climate. The standard applies to existing, new, or retrofit upgrade WWTP projects while collection systems are outside of its scope. The information provided in this Chapter serves as a complement and update to the information provided in *CSA S900.1.18 Climate change adaptation for wastewater treatment plants*. That said, information from the latest edition of *CSA S900.1.18 Climate change adaptation for wastewater treatment plants* has not been re-iterated here. For that reason, practitioners should also refer to the latest edition when conducting a risk assessment and identifying adaptation measures for WWTP projects.

2.4 Projected Climate Changes in Atlantic Canada

In this section, a brief summary of major climate changes in Atlantic Canada is presented. This information is general and should not be used directly in risk assessments which may require more site-specific context and projections.

It is noted that climate projections are updated regularly, and the projections presented herein are reflective of the best available information at the time of publication.

2.4.1 Observed Climate Change in Atlantic Canada

- **Air Temperature:** Annual and seasonal average temperatures are projected to increase across the Atlantic region, with faster rates of change in more northern regions. This will translate to an increase in growing season length throughout Atlantic Canada. The duration and frequency of extreme high temperature events (e.g., number of hot days and heat waves) are also projected to increase, while low temperature extremes are expected to decrease. Changes in the frequency of freeze-thaw cycles will vary depending on the current climate (with most of the region seeing a decrease during the shoulder seasons and an increase or no change in winter). Winter thaw events (warm episodes) are expected to occur more frequently.

- **Water Temperature:** Seasonal increases in surface water temperatures are expected in many areas, although this depends on local conditions. For example, in some locations, air temperatures may drive water temperatures, whereas other locations the water temperature may depend on groundwater inputs.
- **Seasonal Precipitation, Flow Patterns and Drought:** Annual precipitation is expected to increase in the Atlantic region along with the frequency of heavy rainfall events. That said, in many locations the increase in annual precipitation will occur during the winter and spring. Except in parts of Labrador, more precipitation is expected to fall as rain rather than snow (although snowfall extremes may still occur). It is possible that these changes lead to increased occurrence of rain on snow. During the summer and early fall, projections typically show more hot and dry conditions, potentially contributing to longer and more acute droughts, and low flows. It is also expected that regions across Atlantic Canada will experience earlier spring flows from snow melt.
- **Freezing Rain:** The weather systems that cause freezing rain are expected to occur further northward with climate change. This means that certain parts of Atlantic Canada will see an increase in freezing rain, whereas others will see a decrease (e.g., some more southern regions and coasts).
- **Flooding:** According to the Institute for Catastrophic Loss Reduction (ICLR), flooding is the most frequent disaster in Atlantic Canada. Flooding can be caused by different drivers, such as high-intensity precipitation, a river or stream overtopping its banks, ice and debris jams, and extreme coastal water levels. In Atlantic Canada, flooding often occurs due to a combination of these processes. Climate change will lead to increased flooding in many areas, due to changes in high-intensity precipitation, snow melt, rain-on-snow, storm surge, and sea-level rise. It is noted that, because flooding varies depending on local terrain and other factors, it cannot be assessed by projected changes to these climate variables alone (Appendix F for more information).
- **Storm Activity:** Storm intensity and extreme weather events are expected to increase as a result of climate change (e.g., tropical storms, thunderstorms, windstorms, nor'easters). This may lead to more intense precipitation that is combined with surge events, as well as high winds combined with freezing rain and/or blowing snow.
- **Sea Level Rise:** Sea levels have risen globally, as documented by long-term tide gauge records, such as the tide gauge installed in the Halifax Harbour. Relative sea-level change will significantly vary across Atlantic Canada, in part due to vertical land motion. Land subsidence in the south will increase local relative sea level rise, while land rebound in the north will buffer it.
- **Extreme Coastal Water Levels:** Contributors to coastal water levels include sea-level rise, an overall increase in storminess (storm surge), and increased wave action due to reduced ice cover. The extent of impact is highly variable depending on the local coastline.
- **Coastal Erosion.** Coastal erosion is influenced by several factors including the influence of sea level rise, an overall increase in storminess (storm surge), and increased wave action due to reduced ice cover may result in increased inundation and coastal erosion in some areas.
- **Wildfire and Other Disturbance Regimes:** The severity and intensity of natural disturbance regimes is expected to increase under climate change, as a result of forest fires, blow downs/windthrow, precipitation damage, mudslides, invasive species/pathogens/pests, disease, and drought.
- **Permafrost:** Southern Labrador is at the lower end of the permafrost distribution (with isolated, sporadic, and discontinuous permafrost), and Northern Labrador is within the zone of continuous permafrost. Climate change over the next century is expected to have the greatest effect on the more vulnerable permafrost at the southern boundary.

2.5 Climate Impacts

Direct and indirect impacts to the development of wastewater projects and operation of wastewater infrastructure as a result of climate change are described in this section. The impacts of climate change on wastewater systems may include, for example:

- Structural damage and flood risk to buildings and infrastructure.
- Supply chain disruption.
- Power and communication outages.
- Changes in receiving water elevations impacting system hydraulics and outfalls.
- Increase frequency and volume of overflow events and environmental contamination.
- Concentration and temperature changes of influent wastewater impacting biological treatment.
- Breakage and deterioration of wastewater conveyance systems.

Climate impact examples are provided to serve as an illustration of potential considerations and are not all encompassing.

Climate impact examples:

- **Power Outages:** The frequency of power outages may increase with climate change due to projected increases in the severity and frequency of extreme events. Power outages may significantly impact a wastewater system's ability to pump and process wastewater prior to discharge. These impacts are intensified when plants are not equipped with backup power, eliminating the plant's ability to pump or process wastewater completely, and may result with flooding or infrastructure damage within the plant.
- **Coastal Infrastructure:** Impacts of flooding and erosion on coastal infrastructure are likely to increase with climate change. In Atlantic Canada, this may have implications for supply chain along coastal highways and through ship-based transportation. In coastal locations, the salinity of surface water and groundwater may change. Saltwater intrusion facilitated by sea level rise may adversely affect conveyance capacities and lead to increased corrosion of wastewater infrastructure. Sea level rise may lead to increases in groundwater table elevations in coastal areas. Increases in groundwater table elevations may impact sludge management dewatering, cause pipe flotation leading to cracking, and increase infiltration leading to loss of functionality and capacity of the conveyance system.
- **Wet Weather Flows, Hydraulics and Outfalls:** The impacts of flooding, sea level rise, and groundwater elevations on wastewater system hydraulics are expected to increase with climate change. Changes in receiving water levels and flood elevations will impact the location/elevation of outfalls and may result in the need to alternate from gravity flow to a pump system or increased heads for existing pumped systems to facilitate discharge through outfalls, resulting in increased energy requirements. Rising receiving water levels and flood elevations leading to submerged gravity pipelines may lead to sewer back-ups and may contribute to wastewater contamination in receiving water bodies. Flooding may cause a reduction in the service zone of a pump station. High inflows resulting from increased rainfall intensity and extreme precipitation events may impact hydraulic performance of the system and exceed infrastructure capacity. Large precipitation events leading to high flows may cause conveyance issues, such as blockages and breaks, from increased accumulation of debris in pump stations and screens, particularly in combined systems.
- **Overflows:** Increased rainfall intensity, extreme precipitation events, and flooding are likely to lead to increased occurrence of I&I into wastewater systems. Combined systems are the most vulnerable to I&I because they receive storm water overflow. Combined systems will be the source of potential increased frequency and volume of uncontrolled discharges of untreated wastewater to the environment. Increases in

the frequency and volume of wet weather overflow events may result in increased occasions of environmental contamination and result in non-compliance of discharge objectives.

- **Dry Weather Flows:** Seasonal increases in drought conditions or decreased water availability may result in reduced I&I and changes to water usage patterns, such as decreased sanitary water usages. This may reduce flows throughout the system resulting in an increase in wastewater concentration, leading to 'high-strength' wastewater and increased temperatures. High-strength wastewater contains higher pollutant concentrations, including pathogens, which may result in an increased likelihood of blockages, odours, and corrosion of the conveyance system. Influent high strength wastewater may increase the risk of toxicity level exceedances. Submerged outfalls maybe impacted by drought conditions and low/decreasing water levels in receiving waters.
- **Treatment:** Impacts of flooding, high flow events, hot temperatures, and low flows on the management and treatment of wastewater may increase with climate change. Increases in groundwater table elevations may impact sludge management dewatering. Increasing intensity of precipitation events leading to increased storm water runoff and I&I may result in dilution of influent wastewater which can affect biological treatment processes. Influent high strength wastewater, resulting from low flows during drought conditions, poses an increased risk of toxicity levels exceedances and may worsen odours. Performance of biological systems, oxidation ponds and sludge management systems vary with temperature. Potential increases in wastewater temperatures, resulting from low flows, may impact some treatment types, worsen odours and increase corrosion.

It should be noted that the processes involved in changes to runoff potential, flood elevations, and elevation and quality of receiving water bodies are complex and dependent on local factors. Further, the impact of changes to the receiving water quality is highly dependent on the specific WWTP and cannot be considered independently. Nonetheless, these changes may impact approval and compliance criteria for wastewater discharge and treatment objectives.

2.6 Climate Information

This section summarizes important background concepts relating to climate projections. These ideas are key for understanding how climate information is used in climate change risk assessments.

2.6.1 Managing Uncertainty

Decision-makers are already equipped to deal with uncertainty. Weather variability and extreme weather unpredictability is a fact that engineers, planners, emergency service providers, and policymakers have learned to accept and respond to appropriately.

The largest variables surrounding climate change are the rate of change each climate variable will experience over time and the ability to mitigate the change. Given the variability, the available information is best used by applying the full range of projections (from lowest to highest future value) in risk assessment. This is the recommended approach/best practice.

Regardless, the uncertainties associated with climate projections should be factored into, and not against, the use of future climate information. Climate projections and their uncertainties should be framed within the presentation of the data plots and graphs. This allows designers and decision-makers to familiarize themselves with the potential envelope of conditions to be considered.

2.6.2 Understanding Climate Models

A climate model is a computer representation of the climate system. Climate models divide the earth into 3D cells that are 100 to 300 km wide and use equations to simulate atmospheric, oceanic, and other processes. There are over 30 Global Climate Models (GCMs) which are owned by leading scientific institutions around the world and require significant computational power to run.

2.6.3 Understanding the Use of Climate Models

Climate models are used to test how the climate system will behave under future changes. Without climate models, we would need to extrapolate measured historical trends, and this assumes that past processes remain unchanged in the future ("the stationarity assumption"). Considering the climate is a complex system, there can be step-changes and other non-linearities, so the stationarity assumption is often false. Climate models are used to better project the future climate by resolving some of these more complex processes. This is important background information when choosing types and sources of climate information for an assessment (e.g., complementing historical information with climate model projections (Appendix F)).

2.6.4 Emission Scenarios

Greenhouse gases such as carbon dioxide and methane trap the sun's heat within the atmosphere, causing the climate to warm. Emission scenarios represent possible GHG patterns over the 21st century, which depend on population growth, future technology, policies, and conflict. Global climate models are driven by emission scenarios. The Intergovernmental Panel on Climate Change (IPCC) has established several industry-standard scenarios, called Representative Concentration Pathways (RCPs) or Shared Socioeconomic Pathways (SSPs).

2.6.5 Spatial Resolution & Downscaling

The spatial resolution of climate models is important for some processes to be properly modelled. The resolution of GCMs is too coarse to properly resolve certain processes. High-intensity precipitation, for example, is better represented at higher spatial resolutions.

Global climate model outputs can be focused onto smaller areas by using processes referred to as "statistical downscaling" or "dynamic downscaling". Statistical downscaling uses statistical relationships between local climate variables and large-scale predictors, which are then applied to model outputs to approximate local climate projections. Dynamic downscaling uses Regional Climate Models (RCMs), which are driven by outputs of GCMs. Representative concentration pathways are higher-resolution (smaller grid size (e.g., 10 to 50 km)) and include more detailed information on regional conditions (e.g., mountains or water bodies), which increases accuracy. This is important background when choosing types and sources of climate information for an assessment (Section 2.6.12).

2.6.6 Projection Horizons

Projection horizons are the discrete time periods over which climate projections are expressed. These are selected based on the remaining useful life of the program or asset being assessed (e.g., mechanical systems often use a lead-time of 30 years). It is also appropriate to use shorter-term projections to assess the capacity of a project/asset with a long-term planning horizon, as the project/asset will still experience changes over the short term.

Climate projections are expressed as a mean value over a 30-year period, which is compared to a mean over a 30-year baseline (e.g., the 1981-2010). The 30-year periods are to average natural climate variability that occurs on multi-decadal timescales. For example, climate projections for the 2050s are calculated as 2041-2070. This is important background when choosing types and sources of climate information for an assessment (Appendix F).

2.6.7 Climate Indices

The outputs of GCMs and RCMs include variables like temperature, precipitation, humidity, snow, and wind. Indices are calculated from these variables to provide detailed and meaningful projections that can be used by decision-makers (e.g., indices can be threshold-based, minimum, maximum, extremes, or related to duration). Some indices require a combination of variables, such as humidex which involves both humidity and temperature.

2.6.8 Locally-Dependent Climate Changes

While some climate changes are expected to be consistent over a region (e.g., temperature, precipitation, tropical storm intensity, and sea-level rise), others depend more strongly on local processes. For example, changes in water quality depend on the characteristics of the watershed and waterbody (e.g., degree of impact from flushing events and disturbance, stratification, and receiving water quality and dilution).

Flooding extents and depths also depend on the local terrain (e.g., watershed size and width of the riverbed), hydraulic structures (e.g., culverts), and other factors. This means that a given increase in precipitation intensity may result in an amplified increase in flooding in one location, while causing a minimum increase in flooding in another.

In other words, locally dependent climate changes cannot be projected without additional modelling or analysis. In the case of flooding, engineers use impact models (i.e., hydrologic, hydraulic, and hydrodynamic) to convert climate projections for precipitation intensity and extreme coastal water levels into projected changes in in floodplains which are depicted on hazard maps.

2.6.9 Model Ensembles

Climate models use approximations in their mathematical formulations. This is because models cannot capture the entire complexity of the climate system. The approximations are a practical solution to account for phenomena that occur at spatial scales smaller than their grid cells. This means that a given model can overestimate or underestimate the actual climate. Considering different models use different approximations, they give different solutions, each of these solutions representing one possible future. Hence, the range of the models' projections covers the range of possibilities (to the extent that they can be modelled). For that reason, an ensemble of models should be used for climate projections.

This approach for managing climate projection uncertainty is called ensemble modelling. The results of ensembles are presented with graphs and plots that indicate the distribution of data so that an assessment can be made with respect to the spread and overall confidence of the modelled result. Typically, climate projections from model ensembles are reported with percentiles of the ensemble as an indication of the uncertainty (e.g., 10^{th}, 50^{th}, and 90^{th} percentiles). This is important background when choosing types and sources of climate information for an assessment (e.g., climate information based on ensembles and that allow for uncertainty characterization).

2.6.10 Sources of Uncertainty

Although climate models are the best tools available to make projections, there are limitations in the models and associated observational datasets and statistical transformations. The number of limitations depend on the variable being projected, the index, and the climate/geography. For example, confidence in temperature projections is typically greater than for other variables such as precipitation or winds; return period precipitation extremes are harder to project than changes in total precipitation.

The relative importance of each source of uncertainty also depends on the timescale considered. Over the timescale of a few decades, natural variability (fluctuations of the climate system that occur even without any changes in GHG concentrations) dominates and can even hide the climate change signal. That said, over longer time horizons, predictions associated with each emission scenario diverge, lessening the relative significance of uncertainty associated with natural variability. The use of more than one emission scenario mitigates this uncertainty by presenting a range of possibilities. This is important background when choosing types and sources of climate information for an assessment (e.g., complementing historical information with climate model projections (Section 2.6.12)).

2.6.11 Evolving Science

Climate science is fast-moving. Models continue to resolve a higher number of processes more accurately, and updated emissions scenarios, new modelling ensembles and better post-processing of climate projections are becoming available. For example, future generations of models will have better spatial resolution (made possible due to an increase in computing power). Nonetheless, some processes will remain difficult to represent in models, and others will still be smaller than the grid resolution (e.g., cloud microphysics). This is important background when choosing types and sources of climate information for an assessment (e.g., updated sources of information (Section 2.6.12)).

2.6.12 Finding Climate Information

Climate information is available from different types of sources, which range from qualitative to quantitative. Examples are provided in this section.

The reader is referred to Appendix F for a detailed description of how the concepts presented in this Chapter (e.g., climate-asset interactions, scope of the risk assessment, and availability of climate information) apply to the selection of climate information.

2.6.12.1 Local, Operator, & Indigenous Knowledge

Knowledge and experience of the people living or working close to the geography or infrastructure being assessed can be an important source of climate information. For example, some phenomena are only known by residents of the area, such as local processes that have not been studied or modelled (e.g., microclimate in a valley). Local residents can help characterize existing problems such as flooding. Infrastructure operators and people familiar with the site may highlight trends (e.g., changes to wind-driven movements of lake ice or algal blooms). Traditional Indigenous knowledge is a valuable source of information that is complementary to other approaches. Examples of obtaining local, operator and Indigenous knowledge include:

- Consultation and engagement with First Nation, Inuit, and Métis Communities and groups.
- Asking residents to validate flood maps at a public open house.
- Conducting a risk workshop with infrastructure operators.

2.6.12.2 Measured Data

There are many types of directly or indirectly measured data that can be used to characterize historical trends (e.g., data from climate station instruments, tide and river gauges, and satellites). In terms of characterizing future changes, extrapolating measured historical trends can be problematic because it assumes that changes do not speed up or slow down. Nonetheless, an understanding of past changes can complement knowledge of climate projections. Examples of measured data include:

- Historical ice charts from the *Canadian Ice Service archive* from the Government of Canada.
- Tide gauge records from the Department of Fisheries and Oceans Canada (DFO).
- Hourly wind speed measurements from Environment and Climate Change Canada stations.
- *Historical Climate Data* from Environment and Climate Change Canada.

2.6.12.3 Climate Model Projections from Data Portals

Quantitative projections can be obtained from data portals. These are user-friendly and include pre-calculated indices (e.g., total annual precipitation, number of days > 30˚C). Several portals include ensembles of statistically downscaled GCMs, and projections can be extracted for one location or visualized on a map. Yearly precipitation extremes are available (e.g., annual maximum 1-day precipitation). Available indices are typically calculated from daily temperature and precipitation projections only, although ClimateData.ca now has an index also based on evapotranspiration (the Standard Precipitation Evapotranspiration Index (SPEI)). The data portals are increasingly more flexible, allowing for custom-made indices (e.g., on ClimateData.ca). Examples of data portals include:

- *Canadian Centre for Climate Services* from the Government of Canada.
- *The Climate Atlas* from the Prairie Climate Centre.
- *Climate Data and Information* from CLIMAtlantic.

2.6.12.4 Project-Specific Computation of Climate Indices

If data portals cannot provide the required post-processing of climate model projections, another approach is to download the projections and calculate what is required offline. For example, statistically downscaled ensembles of GCMs can be obtained in a gridded format and manipulated with coding expertise. This requires more time but means that the outputs can be tailored to the assessment. Some examples include:

- Computation of sector-specific indices (e.g., 2-day or 5-day precipitation).
- Computation of projections over a desired area (e.g., averaged over a watershed or municipality).
- Generation of certain figures and visualizations not available through data portals (e.g., monthly changes shown on the same plot).

2.6.12.5 Clausius-Clapeyron Equation

For projected changes in precipitation intensity, use of temperature scaling through the Clausius-Clapeyron equation has become the de-facto approach, until better approaches are established. This equation describes the ability of warmer atmosphere to hold more water, simplified as a 7% increase in precipitation intensity per increase in °C. Some examples of where this approach is described include:

- *CSA PLUS 4013 Technical Guide- Development, Interpretation and use of rainfall intensity-duration-frequency (IDF) information: Guidelines for Canadian water resources*.
- *Climate-resilient buildings and core public infrastructure 2020: an assessment of the impact of climate change on climatic design data in Canada* from Environment and Climate Change Canada.

2.6.12.6 Downscaling & Post-Processing Tools

Certain tools are available (online or as software) to aid in the generation of climate scenarios. These are different approaches to projecting future climate data for a given locality. Each has its own assumptions and limitations. Some examples are:

- *IDF-CC Tool* from the Western University Canada.
- *Statistical DownScaling Model – Decision Centric* from SDSM.
- *Canadian Extreme Water Level Adaptation Tool (CAN-EWLAT)* from the Bedford Institute of Oceanography.

2.6.12.7 Impact Models

As described in Section 2.6.8, some variables impacted by climate change are more locally dependent. This includes changes in water quality and water elevations. These variables are not directly available from climate models and instead are independently modelled utilizing the climate projection data.

Future changes in these variables are usually characterized from a physical or data-driven impact model that is developed using future climate data. Some examples include:

- Using a hydrological/hydraulic model to project riverine flooding.
- Using a sewer model to project combined sewer overflows.
- Using a wave model to project wave height.
- Using a water quality model to project lake acidification.
- Using a groundwater model to project low flow.
- Using a hydrodynamic model to project dilution in receiving water bodies.

This information usually comes from a standalone assessment.

2.6.12.8 Floodplains & Hazard Mapping

Another source of information is hazard mapping. Hazard maps are particularly relevant for locally dependent impacts of climate change, such as increased water levels. Other hazard maps can include combined sewer overflows or wave heights.

In the case of flooding, flood line mapping in Atlantic Canada is usually conducted by governing bodies such as municipalities. The challenge is when available hazard mapping has not yet been updated to include the impacts of climate change (e.g., projected increase in precipitation intensity and sea-level rise), and flood modelling is outside the scope of the risk assessment.

In these cases, using the largest historical flood on record is not adequate, as this is rarely representative of future flooding likelihood. Likewise, use of a safety factor (e.g., adding 1.0 m for sea level rise) is not recommended, as the future change in flooding can be drastically different from this, and the safety factor provides false confidence in the analysis.

It is recommended that for existing hazard mapping to be used to characterize baseline flooding. Expert judgement should then be applied to estimate a likelihood score for future projection horizons, based on an understanding of projected impacts of climate change and the local hydrology and hydraulics. The risk assessment should flag the floodplain mapping gap as part of the recommendations.

2.6.12.9 Literature

Certain climate phenomena are not well captured by climate models. These include freezing rain and ice storms, extreme winds, hurricanes, and several others. Future changes in these phenomena need to be characterized from a process-based understanding (i.e., how the factors that affect these phenomena are changing), which is typically obtained from the literature (existing reports and peer-reviewed science articles). The literature can also be a source of both quantitative (snow, freezing rain, sea-level rise) and qualitative (hurricanes, fog) projections. Example sources of projections include:

- *Climate-Resilient Buildings and Core Public Infrastructure: an assessment of the impact of climate change on climatic design data in Canada* from Environment and Climate Change Canada.
- *CMIP5 drought projections in Canada based on the Standardized Precipitation Evapotranspiration Index* in by Tam et al. and published in the Canadian Water Resources Journal (p. 44(1), 90-107).
- *Changes in oceans surrounding Canada* (Chapter 7, p. 343-423) written by Greenan et al. and published by the Government of Canada.

2.7 Climate Change Risk Assessments

This section summarizes important background concepts relating to risk assessments that are key for understanding the context and applicability of a risk assessment for various applications. Practitioners should also refer to the latest edition of *CSA S900.1.18 Climate change adaptation for wastewater treatment plants* when conducting a risk assessment for WWTP projects.

2.7.1 Introductory Concepts

Introductory concepts for climate change risk assessments are summarized below:

- **What is a Risk Assessment?** Risk can be defined as the combination of the severity (or consequence) of impacts and probability (or likelihood) of a climatic event occurring. Risk assessment is one part of a larger decision-making process that helps manage the uncertainty from climate data, understand system capabilities and required functionality, consider evolving risk factors, and prioritize areas for adaptation. A risk assessment qualifies or quantifies the likelihood, vulnerability, and consequence of a climate hazard, to arrive at a prioritization of high-risk areas (e.g., infrastructure and operations). The purpose of the risk assessment is to evaluate both existing and evolving risks to aid in prioritizing adaptation actions. An adaptation plan can then be implemented in either the planning, design, or operational phase of an asset.
- **Who is Involved in a Risk Assessment?** A team consisting of a diverse group of participants with relevant specific subject matter expertise are best equipped to conduct a thorough and comprehensive risk assessment. The multidisciplinary team may be called upon at various stages of the design development and planning process to ensure a broad perspective and approach is being applied. Team members may consist of the plant operators (front-line to office staff), Regulatory Authorities, engineers, planners, climate scientists, and emergency management professionals, where applicable. Practitioners are cautioned that proper application of climate data can be a complex process which requires interpretation of data and characterization of uncertainty. Typically, engineers, planners, policy makers, and operators are trained to work with uncertainty. A team with climate projection expertise can interpret and filter the significant volume of available climate information down to a critical set of projections to be considered in the decision-making process. It is recommended that personnel with climate science expertise be involved throughout the entirety of the risk assessment process to ensure data is interpreted and applied correctly to facilitate informed decisions making.
- **What is the Role of Risk Assessment in New Infrastructure Projects?** For proposed new projects, it is recommended that a climate change risk assessment (if needed) be incorporated at the pre-design phase of

work to determine if/how adaptation efforts are to be undertaken throughout design, construction, and operation. This is consistent with the recommendation in Chapter 1 of these Guidelines that the Pre-design Report be completed such that detailed plans and specifications may be developed from it without substantial alteration of concept and basic considerations. The scope of the risk assessment is established through a "screening process" that is undertaken at the pre-consultation phase to determine the level of detail required for project planning purposes. The screening process should determine the scope and boundaries of the risk assessment based on the expected outcomes of the analysis and how the results will feed into the larger decision-making process. A risk assessment is sometimes required for a funding program. For further guidance pertaining to scoping of a risk assessment refer to Section 4.2.1 and 4.4.2 of the latest edition of *CSA S900.1.18 Climate change adaptation for wastewater treatment plants.*

- **What is the Role of Risk Assessment for Existing Systems?** For existing systems, a risk assessment can be completed at any time during the infrastructure life cycle. End of life, poorly maintained, or "undersized" infrastructure is likely to be more susceptible to the impacts of climate change. For that reason, a risk assessment may be prioritized for infrastructure that is at or near the end of its useful life and replacement is imminent, has not met performance expectations, has been negatively impacted by extreme weather in the past, has insufficient capacity for growth, is undergoing an update to planning (compliance, asset management, etc.), and/or is scheduled for capital improvements. Risk assessment is sometimes required for a funding program.
- **What should a Risk Assessment Include?** The System Owner and practitioner may choose to scope the assessment relative to the importance, criticality or complexity of the asset or system (i.e., highly critical assets may warrant a more in-depth analysis), taking into consideration the current phase in the project's life cycle (i.e., from planning to O&M). Further, the risk assessment can be performed municipality-wide, at the asset-class level (e.g., horizontal vs. vertical assets), or on a single existing or proposed asset. Assessments will vary in the degree of precision required and can be executed qualitatively or quantitatively. The anticipated impact(s) or climate-asset interactions may be a driving factor in the precision level required. A risk "screening process" can be undertaken during the pre-consultation phase of a large, proposed capital project to determine if climate change impacts are a concern, and if so, the scope and scale of the risk assessment required.

2.7.2 Screening-Level Assessment

The purpose of the screening-level assessment is to identify potential climate change impacts to the project, determine the System Owner's risk tolerance, and determine the need for a climate change risk assessment in the pre-design phase. If consideration for the impacts of climate change is found to be applicable to the project but cannot be directly addressed due to data gaps (such as a need for climate projections and a more thorough design review), then a risk assessment is recommended at the pre-design phase.

A screening-level assessment provides an opportunity to identify the general scope in which climate change considerations will be incorporated into the project and determine the need for and scope of a risk assessment. The screening-level assessment includes a high-level evaluation to identify asset-climate interactions and potential system/asset vulnerabilities to climate change over its design/service life.

A risk assessment should be undertaken if a regulatory requirement for incorporation of climate change considerations to the project is identified through the screening-level assessment. Further, if climate change vulnerability is identified through the screening-level assessment and the risk exceeds the System Owner's risk tolerance, then a risk assessment should be undertaken. The screening-level assessment should provide clear reasoning in the conclusion that a risk assessment does or does not need to be completed in the pre-design phase of the project. The assessment should also describe the methodology to be used and include all pertinent information and sources consulted in the screening-level assessment process.

2.7.3 Executing a Risk Assessment

Practitioners should also refer to the latest edition of *CSA S900.1.18 Climate change adaptation for wastewater treatment plants* when conducting a risk assessment for WWTP. An industry best practice is to select a risk assessment methodology that is consistent with *ISO 31000 Risk Management-Guidelines* from the International Organization for Standardization (ISO). The selected risk assessment methodology should consider both historical and future climate. There are many different climate change risk assessments methodologies available that generally use either a "top-down" or "bottom-up" approach. The main difference between the two approaches is the sources of information that drive the assessment. In a simplified description, a "top-down" approach is driven by the climate projections. A "bottom-up" approach focuses on the system that is impacted (e.g., asset and ecological) by determining potential future vulnerabilities and/or failure scenarios. Climate projections are then analysed to determine if the previously determined failure scenarios are plausible. With the "bottom-up approach", information from large stakeholder teams is usually a dominant source for determining the scope of the analysis and characterizing risks.

Climate change risk assessment methodologies will typically include the following key components:

- **List of System Components:** A catalogue of the current capabilities, design, and function of the asset/system is developed. This background system review process may be completed in a workshop setting with experienced operators, engineers, and plant managers. Only systems that are impacted by climate change are included in the list. A climate-asset interaction summary matrix, with climate parameters in columns and system components in rows, can be a helpful presentation tool. Refer to the latest edition of *CSA S900.1.18 Climate change adaptation for wastewater treatment plants* (Section 4.2.2, Section 4.5, and Annex C) for further guidance for conducting background review and identifying system components for consideration.
- **Use of Thresholds:** A threshold is defined as the limit past which an infrastructure asset or system component is critically vulnerable. Thresholds guide the types of outputs required from the climate assessment. Thresholds can be based on review of performance history, design criteria, modelling of system performance, identification of key performance indicators, or asset condition.
- **Climate Analysis:** A climate projections analysis includes a quantitative or qualitive measure of the probability (or likelihood) of a particular climate event occurring at the location of interest. Appendix F provides a detailed description of how the concepts presented in this Chapter (e.g., climate-asset interactions, scope of the risk assessment, and availability of climate information) apply to the selection of climate information for the climate projections analysis. The latest edition of *CSA S900.1.18 Climate change adaptation for wastewater treatment plants* (Section 4.3) provides guidance on conducting a climate analysis, and *CSA S900.1.18 Climate change adaptation for wastewater treatment plants* (Section 4.7.3) provides guidance on quantifying probability for assessments of WWTP.
- **Vulnerability and Consequence Analysis:** The vulnerability and consequence assessment considers the degree to which the system is affected when exposed to changes in climate factors, where climate vulnerability is defined as the degree to which a system is affected by climate stressors, and consequence is defined as the outcome, should a given climate change occur. Consequence should be assessed independently from probability/likelihood. Depending on the risk assessment methodology used, vulnerability may be combined with consequence or may stand alone. Vulnerability can be measured in terms of the system's ability to withstand an anticipated consequence (e.g., backup power can mitigate the

impacts of power outages and, therefore, reduce system vulnerability). There are several categories of consequences to consider such as asset damage, financial loss, loss of service, health and safety, and reputation, among others (*Asset Management & Climate Change: Planning for a Future of Uncertainty* by Ali A, Singh A., et al.). The consequences for not meeting desired levels of service should be considered when determining severity. *CSA S900.1.18 Climate change adaptation for wastewater treatment plants* (Section 4.7.4) provides guidance on quantifying severity of consequences of an interaction.

- **Risk Assessment:** The risk analysis categorizes each climate-asset interaction according to the risk levels identified (i.e., quantitative or qualitative scoring). The risk scoring exercise is typically finalized in a workshop setting with a multidisciplinary team. The risk assessment results can be displayed visually in a risk matrix which categorizes risks from low to high. The risk matrix example below (from the *Climate Lens-General Guidance* written by Infrastructure Canada and published by the Government of Canada) presents a qualitative definition for likelihood and consequence. It is noted that that risk assessment is typically an iterative process as the practitioner often revisits certain conclusions or priority scoring based on relative risk comparisons and multidisciplinary discussions with project stakeholders in a workshop format. The outcome of the risk assessment is a ranking of climate interactions from low to high risk, which will inform the adaptation plan priorities. Refer to the latest edition of *CSA S900.1.18 Climate change adaptation for wastewater treatment plants* (Section 4.7) for further guidance on conducting a risk assessment.

Table 2.1: Risk Assessment Matrix

Consequences						
	Very high	Moderate risk	High risk	High risk	Extreme risk	Extreme risk
	High	Low risk	Moderate risk	High risk	High risk	Extreme risk
	Moderate	Low risk	Low risk	Moderate risk	High risk	High risk
	Low	Negligible risk	Low risk	Low Risk	Moderate risk	Moderate risk
	Very low	Negligible risk	Negligible risk	Low risk	Low risk	Low risk
		Very low	Low	Moderate	High	Very high
		Likelihood				

Extreme risk (red): Immediate controls required.
High risk (orange): High priority control measures required.
Moderate risk (yellow): Some controls required to reduce risks to lower levels.
Low risk (blue): Controls likely not required.
Negligible risk (green): Risk events do not require further consideration.

2.8 Climate Change Adaptation

Adaptation planning involves identifying priority options to improve resiliency to climate risks. Risks are first evaluated and ranked through the risk assessment process. Adaptation measures, sometimes referred to as risk treatment measures, are a set of actions designed to reduce the risk of negative impacts (and potential loss) to an acceptable level. Adaptation also includes taking advantage of opportunities presented with climate change. An adaptation plan is the collection of preferred risk treatment measures into a strategy document, which typically outlines the cost or level of effort associated with each measure, as well as the timelines and parties responsible for implementation. An adaptation plan can be implemented in either the planning, design, or operational phase of an infrastructure asset.

Adaptation measures are first identified for each unacceptable risk, which is either guided by the System Owner's risk tolerance or based on risk ranking from the risk assessment methodology (i.e., high risk categories may require risk treatment measures). There are many potential adaptation measures that can be explored, some of which may include:

- Developing Emergency Response Plans (ERPs) or Contingency Plans (CPs).
- Adjusting operating practices.
- Upgrading infrastructure.
- Creating new practices or processes.
- Monitoring performance overtime and re-evaluating as needed.
- Implementing an advanced warning system.
- Adapting engineering design parameters or safety factors.
- Altering maintenance plans.
- Relocating or raising the asset beyond the future flood limits.
- Completing a more detailed analysis to address data gaps.

A "do nothing" or a "business as usual" approach is also an option, however, this may not be appropriate for high-risk interactions.

Once a list of potential adaptation measures is finalized, the preferred measures are selected, for example by taking into consideration the System Owner's strategic goals, stakeholder expectations, and business level/service/operational targets. One tool for selecting the preferred adaptation measure, or assessing the overall feasibility, is a cost-benefit analysis. This analysis quantifies the costs of the risk treatment measure against the measure's benefits, which may include improved safety, reduced service disruptions, or decreased environmental contamination. Lifecycle cost-benefit analysis is used to determine the set of investments with the lowest Net Present Value (NPV). To quantify the feasibility of the proposed risk treatment measures, *Climate Lens-General Guidance* written by Infrastructure Canada and published by the Government of Canada provides a methodology for calculating the loss avoided and return on investment of the proposed adaptation measure.

No- or Low-regret adaptation measures are those that have a proven cost-benefit regardless of climate change, or under all future climate scenarios. For climate interactions with a lower risk profile in the short-term, practitioners may opt to monitor the asset performance over time and adjust the adaptation plan accordingly as the risk profile increases, or as new technologies and information becomes available. Alternatively, if action must be taken in the short term, leaving available space for flexibility will allow the system to be expanded or built upon at a later time.

Selected adaptation measures should address the risk directly and not transfer downstream to another System Owner or asset(s). Considerations for adaptation measures with multiple benefits, such as those that reduce GHG emissions, consider the application of natural infrastructure, or improve environmental and social conditions are preferred.

2.9 Documentation

As previously outlined in Chapter 1, practitioners should provide a screening-level assessment or risk assessment (if applicable) as part of the Pre-design Report submission. The screening-level assessment or risk assessment (if applicable) should also be provided within the Detailed Design Report as it is intended to be a stand-alone document.

The latest edition of *CSA S900.1.18 Climate change adaptation for wastewater treatment plants* (Section 4.9) provides guidance for elements to be included in the submitted report. Refer to the latest edition of *CSA S900.1.18 Climate change adaptation for wastewater treatment plants* (Annex F) for examples of a summary risk assessment report for WWTP projects.

2.10 References

Abeysirigunawardena, Dilumie S., Dan J. Smith and Bill Taylor. “Extreme Sea Surge Responses to Climate Variability in Coastal British Columbia, Canada. Vol. 101. 5. Ed. Annals of the Association of American Geographers”. Taylor & Francis Online, 1 August 2011. 992-1010. <https://www.tandfonline.com/doi/abs/10.1080/00045608.2011.585929>.

Agilan, V. and N.V. Umamahesh. "Advances in Water Resources." *What are the best covariates for developing non-stationary rainfall Intensity-Duration-Frequency relationship?* Vol. 101. Warangal: Department of Civil Engineering, National Institute of Technology, March 2017. 11-22. <https://www.sciencedirect.com/science/article/pii/S0309170816308211>.

Ali A., Singh A. *Asset Management & Climate Change: Planning for a Future of Uncertainty*. WEAO 2019 Technical Conference, Toronto, Ontario, 2019.

Assembly of First Nations, Environmental Stewardship Unit. *Climate Change and Water: Impacts and Adaptations for First Nations Communities*. Ottawa: Funded by Natural Resources Canada: Climate Change Impacts and Adaptation Program (CCIAP), March 2008. <https://www.afn.ca/uploads/files/env/08-03-27_climate_change_and_water_research_paper_final.pdf>.

Atkinson, David E., Donald L. Forbes and Thomas S. James. "Canada's Marine Coasts in a Changing Climate." *Dynamic coasts in a changing climate*. Ed. Donald S. Lemmen, et al. Ottawa: Government of Canada, April 2016. 22-68, Chapter 2. <https://www.nrcan.gc.ca/sites/www.nrcan.gc.ca/files/earthsciences/pdf/assess/2016/Coastal_Assessment_FullReport.pdf>.

Attema, J.J., J.M., Lenderink, G. Loriaux and E.V. Meijgaard. “Linking increases in hourly precipitation extremes to atmospheric temperature and moisture changes”. Vol. 5. 2. De Bilt: IOP Publishing Ltd, Environmental Research Letters, 16 April 2010. <https://iopscience.iop.org/article/10.1088/1748-9326/5/2/025208>.

Bélanger, Diane, et al. Human Health in a Changing Climate: A Canadian Assessment of Vulnerabilities and Adaptive Capacity. Ed. Jacinthe Séguin. Ottawa: Authority of the Minister of Health, 2008. <https://cdn.dal.ca/content/dam/dalhousie/pdf/faculty/science/emaychair/Reports%20Section/Emay_HumanHealthChangClim.pdf>.

Bourque, Alain, et al. *From Impacts to Adaptation: Canada in a Changing Climate 2007*. Ed. Donald S., Warren, Fiona J. Lemmen, Jacinthe Lacroix and Elizabeth Bush. Ottawa:

Boyle, Jessica, Maxine Cunningham and Julie Dekens. "IISD Report." *Climate Change Adaptation and Canadian Infrastructure: A Review of the Literature.* Winnipeg: International Institute for Sustainable Development, November 2013. <https://www.iisd.org/system/files/publications/adaptation_can_infrastructure.pdf>.

Brandes, D, J.G. Hoffmann and James, T. Mangarillo. *Base Flow Recession Rates, Low Flows, and Hydrologic Features of Small Watersheds in Pennsylvania, USA*. Vol. 41. 5. Journal of The American Water Resources

Association, 1 October 2005. 1177-1186. <https://www.semanticscholar.org/paper/Base-Flow-Recession-Rates%2C-Low-Flows%2C-and-Features-Brandes-Hoffmann/501ea5458128f215de9fcb31e4c3687f267c7af3>.

Brown, Ross D., and Robert O. Braaten. "Spatial and temporal variability of Canadian monthly snow depths, 1946-1995". Vols. 36, 1998. 1. Atmosphere-Ocean, 1 August 1997. 37-54. <https://www.tandfonline.com/doi/pdf/10.1080/07055900.1998.9649605>.

Bush, E. and G. Flato. CCCR 2019 *Canada's Changing Climate Report*. Ed. E. Bush and D. S. Lemmen. Ottawa: Government of Canada, 2019. <https://www.nrcan.gc.ca/sites/www.nrcan.gc.ca/files/energy/Climate-change/pdf/CCCR_FULLREPORT-EN-FINAL.pdf>.

Canadian Institute of International Affairs (CIIA). *The Transboundary Water Resources of Canada and the United States*. CIAA: Toronto, 2005.

Canadian Standards Association. *CSA S900.1-18 Climate change adaptation for wastewater treatment plants*. Toronto: CSA group, December 2018.

Cannon, A.J. *Climate-resilient buildings and core public infrastructure 2020: an assessment of the impact of climate change on climatic design data in Canada*. Environment and Climate Change Canada, 2020. <https://publications.gc.ca/site/eng/9.893021/publication.html>.

Climate Atlas of Canada. *Data Sources and Methods*. 2019. Accessed February 2020. <https://climateatlas.ca/data-sources-and-methods>.

Climate Data Canada. n.d. Climate Data for Resilient Canada. Accessed 2020-2021 <https://climatedata.ca/>. Climate Ready Water Utilities. *Adaptation Strategies Guide for Water Utilities.* EPA 817-K-15-001. Washington: United States Environmental Protection Agency, Office of Water (4608-T), February 2015. <https://www.epa.gov/sites/default/files/2015-04/documents/updated_adaptation_strategies_guide_for_water_utilities.pdf>.

Conradt, Tobias, et al. "Hydrological Sciences Journal." *Measured effects of new lake surfaces on regional precipitation*. Vol. 52. 5. Taylor & Francis Online, 7 August 2007. 936-955.

Dessai, Suraje and Mike Hulme. "Climate Policy." *Does climate adaptation policy need probabilities?* Vol. 4. 2. Taylor & Francis Online, 21 July 2004. 107-128. <https://www.tandfonline.com/doi/abs/10.1080/14693062.2004.9685515>.

Environment and Natural Resources. *Canadian Climate Normals*. Government of Canada. March 2022. <https://climate.weather.gc.ca/climate_normals/index_e.html>

Environment and Natural Resources. *Historical Climate Data*. Government of Canada, March 2022. <https://climate.weather.gc.ca/>

Environment and Natural Resources. *Ice Forecasts and Observations*. Government of Canada, May 2019. <https://www.canada.ca/en/environment-climate-change/services/ice-forecasts-observations.html>

Field, C.B., Working Group II Contribution to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change. C*limate Change 2007: Impacts, Adaptation, and Vulnerability*. Ed. Martin Parry, et al. New York: Cambridge University Press for the Intergovernmental Panel on Climate Change, 2007. <https://www.ipcc.ch/site/assets/uploads/2018/03/ar4_wg2_full_report.pdf>.

Fisheries and Oceans Canada. *Tides, currents, and water levels*. Government of Canada, January 2022. <https://www.tides.gc.ca/en/tides-currents-and-water-levels>

Gan, Thian Y. *Reducing Vulnerability of Water Resources of Canadian Prairies to Potential Droughts and Possible Climatic Warming.* Water Resources Management: Vol. 14. 2. April 2000. 111-135. <https://www.researchgate.net/publication/226303017_Reducing_Vulnerability_of_Water_Resources_of_Canadian_Prairies_to_Potential_Droughts_and_Possible_Climatic_Warming>.

Government of Australia. *Approaches to Climate Change Impact Assessment.* Accessed 24 December 2020. Government of Australia. February 2020. <https://www.climatechangeinaustralia.gov.au/en/learning-support/using-projections/approaches-climate-change-impact-assessment/>.

Government of Canada, 2008. 448. <https://www.nrcan.gc.ca/sites/www.nrcan.gc.ca/files/earthsciences/pdf/assess/2007/pdf/full-complet_e.pdf>.

Greenan, B.J.W., et al. "Bush and Lemmen, Canada's Changing Climate Report." *Changes in Oceans Surrounding Canada*. Ottawa: Government of Canada, 2019. 343-423, Chapter 7 in (eds.). <https://www.nrcan.gc.ca/sites/www.nrcan.gc.ca/files/energy/Climate-change/pdf/CCCR_Chapter7-oceans.pdf>.

Hallegatte, Stephane, Jun Rentschler and Julie Rozenberg. *Adaptation Principles: A Guide for Designing Strategies for Climate Change Adaptation and Resilience*. Washington: World Bank Group, 17 11 2020. <https://openknowledge.worldbank.org/handle/10986/34780 License: CC BY 3.0 IGO>.

Hurd, Brian, et al. "Journal of the American Water Resources Association." *Relative Regional Vulnerability of Water Resources to Climate Change*. Vol. 35. 6. Woodbridge: American Water Resources Association, 8 June 2007. 1399-1409. <https://onlinelibrary.wiley.com/doi/abs/10.1111/j.1752-1688.1999.tb04224.x>.

Infrastructure Canada. *Climate Lens – General Guidance*. Government of Canada. October 2019. <https://www.infrastructure.gc.ca/pub/other-autre/cl-occ-eng.html>

Infrastructure Canada. *Climate Lens – General Guidance*. Infrastructure Canada, 2020. <https://www.infrastructure.gc.ca/pub/other-autre/cl-occ-eng.html>.

Infrastructure Canada. *Climate Resilient Buildings and Core Public Infrastructure Initiative.* Government of Canada, 27 October 2020. 10 January 2021. <https://www.infrastructure.gc.ca/plan/crbcpi-irccipb-eng.html>.

Institute for Catastrophic Loss Reduction, Climate Risk Institute, and Deutsche Gesellschaft für Internationale Zusammenarbeit. PIEVC Engineering Protocol. n.d. February 2020. <https://pievc.ca/protocol/>.

Intergovernmental Panel on Climate Change. *AR6 Climate change 2021: The Physical Science Basis*. IPCC, 2021. <https://www.ipcc.ch/report/ar6/wg1/>.

International Organization for Standardization (ISO). *ISO 31000 Risk Management – Guidelines*. Switzerland: ISO, 2018.

IPCC, 2007: *Climate Change 2007: Impacts, Adaptation and Vulnerability*. Contribution of Working Group II to the Fourth Assessment Report of the Intergovernmental Panel on Climate Change, M.L. Parry, O.F. Canziani, J.P. Palutikof, P.J. van der Linden and C.E. Hanson, Eds., Cambridge University Press, Cambridge, UK, 976pp. <https://www.ipcc.ch/site/assets/uploads/2018/03/ar4-wg2-intro.pdf>.

James, T.S., et al. *Relative Sea-level Projections in Canada and the Adjacent Mainland United States*. Geological Survey of Canada, Open File 7737. Natural Resources Canada, 2014. <https://northsaanich.ca/wp-content/uploads/Relative-Sea-level-Project-in-Canada-Adjacent-Mainland-US.pdf>.

Kharin, V. V., et al. "Climatic Change." *Changes in temperature and precipitation extremes in the CMIP5 ensemble*. Vol. 119. Springer, 20 February 2013. 345-357. <https://link.springer.com/article/10.1007%2Fs10584-013-0705-8>.

Li, G., Zhang, X., Cannon, A. J., Murdock, T., Sobie, S., Zwiers, F., … Qian, B. "Indices of Canada's future climate for general and agricultural adaptation applications". Climatic Change, 2018. 148(1–2), 249–263. https://doi.org/10.1007/s10584-018-2199-x

Mehdi, B.B. et al. *Impacts of Climate Change on Canadian Water Resources*. Brace Centre for Water Resources Management: Ste. Anne de Bellevue, Quebec, 2007. [online: NRCan: <www.c-ciarn.ca/pdf/ciarn_cwra.pdf>.

Mudryk, L. R., Derksen, C., et al. "The Cryosphere." *Canadian snow and sea ice: historical trends and projections*. Vol. 12. 4. European Geosciences Union, 4 April 2018. 1157-1176. <https://tc.copernicus.org/articles/12/1157/2018/>.

Natural Resources Canada. *Canada's Marine Coasts in a Changing Climate.* Ed. Donald S. Lemmen, et al. Ottawa: Government of Canada, 2016. 274. <https://www.nrcan.gc.ca/sites/www.nrcan.gc.ca/files/earthsciences/files/pdf/NRCAN_fullBook%20%20accessible.pdf>.

Nelitz, Marc, et al. *Tools for Climate Change Vulnerability Assessments for Watersheds*. Vancouver and Kelowna: ESSA Technologies Ltd., Canadian Council of Ministers of the Environment, 26 August 2013. <https://mynanbo.org/public/documents/outils/uploaded/nxdzgA3x.pdf>.

Nova Scotia Museum of Natural History. "Natural History of Nova Scotia." T*he Dynamics of Nova Scotia's Climate*. Vol. 1. Ed. Derek Davis and Sue Browne. Halifax: Nimbus Publishing, 1996. <file:///C:/Users/amogensen/Downloads/3752-5657-1-SM.pdf>.

Sharma, Ashish, Conrad Wasko and Dennis P. Lettenmaier. "Water Resources Research." *If Precipitation Extremes Are Increasing, Why Aren't Floods?* Vol. 54. 11. Washington: AGU Advancing Earth and Space Science, 5 November 2018. 8545-8551. <https://agupubs.onlinelibrary.wiley.com/doi/full/10.1029/2018WR023749>.

Sospedra-Alfonso, Reinel and William J. Merryfield. "*Influences of Temperature and Precipitation on Historical and Future Snowpack Variability over the Northern Hemisphere in the Second Generation Canadian Earth System Model"*. Vol. 30. 12. Victoria: Journal of Climate, 15 June 2017. 4633-4656. <https://journals.ametsoc.org/view/journals/clim/30/12/jcli-d-16-0612.1.xml>.

Standards Council of Canada. *CSA W204:19 Flood Resilient Design of New Residential Communities*. Toronto: CSA Group, December 2019. <https://www.csagroup.org/store/product/CSA%20W204%3A19/>.

The Institute for Catastrophic Loss Reduction (ICLR). *Telling the Weather Story*. Toronto: Insurance Bureau of Canada, June 2012. <https://www.iclr.org/wp-content/uploads/PDFS/telling-the-weather-story.pdf>.

U.S. Environmental Protection Agency. *Climate Change Adaptation Plan.* Washington: EPA 100-K-14-001, June 2014. <https://www.epa.gov/sites/default/files/2015-08/documents/adaptationplans2014_508.pdf>.

U.S. Environmental Protection Agency. *Wildfires: How Do They Affect Our Water Supplies?* 13 August 2019. <https://www.epa.gov/sciencematters/wildfires-how-do-they-affect-our-water-supplies>.

United States Agency International Development. *A Guide for USAID Project Managers Potable Water, Incorporating Climate Change Adaptation in Infrastructure Planning and Design*. Global Climate Change, Adaptation, And Infrastructure Issues Knowledge Management Support, Contract No. EDH -I-00-08-00023-00, Order No. OAA -TO-12-00057, 2015. <http://businessdocbox.com/Green_Solutions/69174953-Incorporating-climate-change-adaptation-in-infrastructure-planning-and-design.html>.

Vincent, L. A., et al. “Changes in Canada's Climate: Trends in Indices Based on Daily Temperature and Precipitation Data”. Vol. 56. 5. Atmosphere Ocean, 7 July 2018. <https://www.tandfonline.com/doi/full/10.1080/07055900.2018.1514579>.

Way, Robert G and Antoni G. Lewkowiz. “Environmental controls on ground temperature and permafrost in Labrador, northeast Canada”. Wiley Online Library, 11 March 2018. 29(2), 73-85. <https://onlinelibrary.wiley.com/doi/10.1002/ppp.1972>.

World Bank. “Climate & Disaster Risk Screening Tool”. The World Bank, IBRD, IDA, 2019. <https://climatescreeningtools.worldbank.org/>.

Zhang, Yu; Chen, Wenjun; Riseborough, Daniel W. “Disequilibrium response of permafrost thaw to climate warming in Canada over 1850–2100”. Vol. 35. 2. AGU Advancing Earth and Space Science, 16 January 2008. 2-5.

Table of Contents

Chapter 3 Design of Wastewater Sewers

The latest edition of *CSA W204:19 Flood Resilient Design of New Residential Communities* should be referenced when designing wastewater sewers.

3.1 Types of Wastewater Collection Systems

In general, and except for special circumstances, the Regulatory Authority will approve plans for new systems or extensions only when designed as separated sewer systems in which precipitation from roofs, streets, and other areas and groundwater from foundation drains are excluded. Overflows from intercepting sewers fall under the national standards provided in the *Canada-wide Strategy for the Management of Municipal Wastewater Effluent* from the Canadian Council of Ministers of the Environment (CCME). The national standards state that overflows due to development are not permitted to increase in frequency, volume, or duration unless this occurs as part of an approved long-term management plan. Otherwise, provision should be made for treating the overflow. Instances of overflow events may become more frequent due to increases in the intensity and frequency large precipitation events. Under the *Fisheries Act* by Fisheries and Oceans Canada and published by the Government of Canada, the discharge of sanitary and/or combined sanitary and storm water is considered a deleterious substance.

3.2 Design Capacity Considerations

In general, wastewater systems should be designed for the estimated ultimate tributary population, except in considering parts of the systems that can be readily increased in capacity. Similarly, consideration should be given to the maximum anticipated capacity requirements of institutions, industrial parks, etc.

In determining the required capacities of wastewater sewers, the following factors should be considered:

- Maximum hourly domestic wastewater volume.
- Additional maximum wastewater volume from industrial plants.
- Inflow and groundwater infiltration.
- Tidal infiltration including climate change considerations (e.g., sea level rise and storm surge).
- Topography of area.
- Location of WWTP.
- Depth of excavation.
- Pumping requirements.
- Combined sewer overflows.

The basis of design for all sewer projects should accompany the documents.

3.3 Hydraulic Design

3.3.1 Wastewater Flows

Wastewater flows are made up of waste discharges from residential, commercial, institutional, and industrial establishments, as well as extraneous non-waste flow contributions such as groundwater and surface runoff entering the wastewater collection system. Whenever practical, flow monitoring should be conducted to determine hydraulic requirements; especially if the system is a combined storm sewer and sanitary system. In this case, it is recommended to include climate change considerations (e.g., changes to flooding, sea level rise, and increases in the intensity and frequency of precipitation events) to determine hydraulic requirements. In the absence of flow monitoring data, the design should consider the methodology provided in the following sections.

3.3.2 Extraneous Wastewater Flows

3.3.2.1 Inflow

When designing sanitary wastewater systems, allowances must be made for the leakage of groundwater into the sewers and building sewer connections (infiltration), and for other extraneous water entering the sewers from such sources as leakage through manhole covers, foundation drains, roof down spouts, etc. (inflow).

Due to the extremely high peak flows that can result from roof down spouts, they should not, in any circumstances, be connected directly or indirectly via foundation drains to sanitary wastewater sewers. Similarly, the connection of foundation drains to sanitary sewers should not be permitted. Studies have shown that flows from this source can result in gross overloading of wastewater sewers, pumping stations, and WWTPs for extended periods of time. It is recommended that foundation drainage be directed either to the surface of the ground, a storm sewer system (if one exists), or a subsurface infiltration field or chamber if applicable.

3.3.2.2 Infiltration

The amount of groundwater leakage directly into the sewer system (infiltration) will vary with the age of the system, quality of construction, type of joints, ground conditions, level of groundwater in relation to pipe, etc. Although such infiltration can be reduced by proper design and construction, it cannot be eliminated, and an allowance must be made in the design to accommodate infiltration. Despite the fact that these allowances are referred to as infiltration allowances, they are intended to cover the peak extraneous flows from all sources likely to contribute non-waste flows to the wastewater sewer system. The infiltration allowances used for wastewater sewer design, however, should not be confused with leakage limits used for acceptance testing following construction. The latter allowances are significantly lower and apply to a wastewater sewer system when the system is new and without the private property portions of the building sewers constructed.

3.3.2.3 Extraneous Wastewater Flows

In computing the total peak flowrates for design of sanitary sewers, the designer should include allowances as specified below to account for flow from extraneous sources.

3.3.2.3.1 General Inflow/Infiltration Allowance

A general I&I allowance should be applied, irrespective of land use classification, to account for wet-weather inflow to manholes not located in street sags and for infiltration flow into pipes and manholes. In addition, a separate allowance for inflow to manholes located in street sags should be added as per the following sections. Structures that may be in flood prone or low-lying areas, including climate change considerations, should also be considered for a separate inflow allowance:

- The area allowance ranges from 0.14 to 0.3 L/s per gross ha. Climate change should be considered in determining this value. Refer to Chapter 2 for climate change considerations.

3.3.2.3.2 Manholes in Sag Locations

When sanitary sewer manholes are located within roadway sags or other low areas (and are thus subject to inundation during major rainfall events) the sanitary design peak flowrate should be increased by 0.4 L/s for each such manhole. For new construction, all sanitary manholes and grade rings in sag locations are to be waterproofed and should be fitted with lid seals.

For planning purposes and downstream system design, where specific requirements for an area are unknown, the designer should make a conservative estimate of the number of such manholes which may be installed in the contributing area based on the nature of the anticipated development and include an appropriate allowance in the design.

3.3.3 Domestic Wastewater Flows

In the absence of conducting flow monitoring to determine actual flowrates, the following criteria can be used in determining peak wastewater flows from residential areas, including single and multiple housing, mobile home parks, etc.:

- Design population derived from drainage area and expected maximum population over the design period.
- Average daily domestic flow (exclusive of extraneous flows) of 380 L/cap·d.
- Peak extraneous flow (including peak I&I).
- Peak domestic wastewater flows to be calculated by the following equation:

$$Q(d) = \frac{PqM}{86.4} + \frac{(IA \text{ or } i \sum DL)}{86.4} + SN$$

Where:

Q(d) = Peak domestic wastewater flow, including extraneous flow (in L/s).
P = Design population (in thousands).
q = Average daily per capita domestic flow (in L/cap·d), exclusive of extraneous flows.
M = Peaking factor (as derived from the following formulas).

Harman Formula

$$M = 1 + \frac{14}{4 + P^{0.5}}$$

or

Babbit Formula

$$M = \frac{5}{P^{0.2}}$$

Or as determined from flow studies for similar developments in the same municipality). The minimum permissible peaking factor should be 2.0.

I = Unit of peak extraneous flow (in L/s per ha).
A = Tributary area (in gross ha).
i = Unit of peak extraneous flow (in m^3/cm of pipe diameter/km length of pipe/day).
D = Diameter of pipe (in cm).
L = Length of pipe (in km).
S = Unit of manhole inflow allowance for each manhole in sag location (in L/s).
N = Number of manholes in sag locations.

3.3.4 Commercial & Institutional Wastewater Flows

3.3.4.1 Flow Variation

The wastewater flow from commercial and institutional establishments varies greatly with the type of facilities in the development, the population using the facilities, the presence of water metering, the extent of extraneous flows entering the sewers, etc.

3.3.4.2 Flow Equivalent

In general, the method of estimating wastewater flows for large commercial areas is to estimate a Population Equivalent (PE) for the area covered by the development and then calculate the wastewater flows on the same basis in the previous section. A PE of 85 persons per ha is often used. It is also necessary to calculate an appropriate peaking factor and select a representative unit of peak extraneous flow.

3.3.4.3 Individual Flowrate

For individual commercial and institutional users, the wastewater flowrates in Table 3.1 are commonly used for design. Note that jurisdictions may have their own guidelines to use for design.

Table 3.1: Wastewater Flows (Average Daily)

Type of Establishment		(L/day)
Residence	Private dwelling	380 per person
	Apartment building	380 per person
Transient Dwelling Units	Hotels	380 per bedroom
	Lodging houses and tourist homes	270 per bedroom
	Motels and tourist cabins	300 per bedroom (add for restaurant)
Industrial and Commercial Buildings	Does not include process water or cafeteria	45 per employee
	With showers	90 per employee
Camps	Campsite	500 per campsite
	Trailer camps (private bath)	380 per person
	Trailer camp (central bath, etc.)	230 per person
	Trailer camp (central bath, laundry)	300 per person
	Luxury camps (private bath)	380 per person
	Children’s camps (central bath, etc.)	230 per person
	Labour camps	225 per person
	Day camps (no meals)	70 per person
Restaurants (including washrooms)	Average type (2 x fire commissioner’s capacity)	225 per seat + 100 per employee
	Bar/cocktail lounge (2 x fire commissioner’s capacity)	25 per patron
	Short order or drive-In service	25 per patron
	24 -hour	225 per seat
	Non 24-hour	160 per seat
Clubhouses	Residential type	380 per person
	Non-residential type (serving meals)	160 per person

Type of Establishment		(L/day)
	Golf club	40 per member
	Golf club (with bar and restaurant)	115 per seat
Institutions	Hospitals	950 per bed
	Other institutions	450 per resident
Schools	Basic	50 per person
	With cafeteria	70 per person
	With cafeteria and showers	90 per person
	With cafeteria, showers, and laboratories	115 per person
	Boarding	380 per person
Theatres	Theatre (indoor)	25 per seat
	Theatre (drive-in with food stand)	25 per car
Automobile Service Stations	No car washing	20 per car served
	Car washing	340 per car washed
Miscellaneous	Stores, shopping centres, and office buildings	6 per m^2
	Factories (8-hour shift)	115 per person
	Self-service laundries	1,800 per machine
	Bowling alleys	900 per alley
	Swimming pools and beaches	70 per person
	Picnic parks (with flush toilets)	50 per person
	Fairgrounds (based upon average attendance)	25 per person
	Assembly halls	35 per seat
	Airports (based on passenger use)	15 per passenger
	Churches (no kitchen)	25 per seat
	Churches (with kitchen)	35 per seat
	Beauty parlours	200 per seat
	Barber shops	75 per seat
	Hockey rinks	15 per seat
	Daycare centre	115 per child
	Liquor licence establishments	115 per seat
	Mobile home parks	1,350 per space
	Nursing and rest homes	450 per resident
	Senior citizen home	600 per apartment
	Recreational vehicle park	180 per space

3.3.4.4 Peak Factor

When using the above unit demands, maximum day and peak rate factors must be developed. For establishments in operation for only a portion of the day (such as schools, shopping plazas, etc.), the water usage should also be factored accordingly. For instance, with schools operating for 8 hours per day, the water usage rate could be an average rate of 70 L/student-day x 24/8 or 210 L/student day over the 8-hour period of operation. The water usage will drop to residual usage rates during the remainder of the day. Schools generally do not exhibit large maximum day to average day ratios and a factor 1.5 will generally cover this variation. For estimation of peak demand rates, an assessment of the water using fixtures is generally necessary and a fixture-unit approach is often used.

The peak water usage rates in campgrounds will vary with the type of facilities provided (showers, flush toilets, clothes washers, etc.) and the ratio of these facilities to the number of campsites. A peak rate factor of 4 will generally be adequate, however, and this factor should be applied to the average expected water usage at full occupancy of the campsite.

3.3.5 Industrial Wastewater Flows

3.3.5.1 Flow Variation

Peak wastewater flowrates from industrial areas vary greatly depending on such factors as the extent of the area, the types of industries present, the provision of in-plant treatment or regulation of flows, and the presence of cooling waters in the sanitary sewer system. Utilities should refer to the sewer use bylaws or requirements by local authorities to determine discharge requirements for the industry.

3.3.5.2 Flowrate

The calculation of design wastewater flowrates for industrial areas is difficult. Careful control over the type of industry permitted in new areas is the most acceptable way to approach the problem. In this way, a reasonable allowance can be made for peak industrial wastewater flow for an area and then the industries permitted to locate in the area can be carefully monitored to ensure that all the overall allowances are not exceeded. Industries with the potential to discharge wastewater at higher than the accepted rates could be required to provide flow equalization and/or off-peak discharge facilities or be restricted by a sewer-use by-law.

3.3.5.3 Flow Allowances

Some typical wastewater flow allowances for industrial areas are 35 m^3/ha/day for light industry and 55 m^3/ha/day for heavy industry.

3.3.6 Combined Sewer Interceptors

In addition to the above requirements, interceptors for combined sewers should have capacity to receive a sufficient quantity of combined wastewater for transport to treatment works to ensure attainment of the appropriate provincial and federal water quality standards.

3.3.6.1 Combined Sewer Overflows

Combined Sewer Systems (CSSs) are wastewater collection systems that transport both sanitary wastewater and storm water in a single pipe to a treatment plant. During periods of heavy rainfall or wet weather, the capacity of the CSS and/or treatment plant may be exceeded resulting in direct discharges of untreated wastewater to receiving environments. These overflows are referred to as CSOs. Requirements for CSO treatment should be as specified by the Regulatory Authority having jurisdiction.

The design requirements for sanitary sewers outlined in these Guidelines specify that all new wastewater systems be designed as separate systems. There will remain many existing CSSs. This will result in the continued existence of CSOs. Overflows from intercepting sewers fall under the national standards provided in the *Canada-wide Strategy for the Management of Municipal Wastewater Effluent* from the Canadian Council of Ministers of the Environment. The national standards state that overflows due to development are not permitted to increase in frequency, volume, or duration unless this occurs as part of an approved long-term management plan. Otherwise, provision should be made for treating the overflow. Instances of overflow events may become more frequent due to increases in the intensity and frequency large precipitation events. Under the *Fisheries Act* by Fisheries and Oceans Canada and published by the Government of Canada, the discharge of sanitary and/or combined sanitary and storm water is considered a deleterious substance.

Reducing the frequency, volume, and duration of CSOs is necessary to facilitate new development. By offsetting the added development flows by removing an equivalent amount of storm water impacts in the sanitary and/or combined sewers, the requirements of the *Canada-wide Strategy for the Management of Municipal Wastewater Effluent* from the Canadian Council of Ministers of the Environment national standards can be met. To achieve this, control measures downstream of the excess capacity typically are used. These include the following:

- Collection system inspection and removal of obstructions.
- Tide and control gate maintenance, repair, and replacement.
- Regular installation and adjustment.
- Reduction/retardation of inflows and infiltration.
- Upgrade and adjustment of pumps.
- Raising existing weirs and installation of new weirs.
- System of real-time monitoring/network.

Figure 3.1 classifies some of the various types of regulating structures for outlet control.

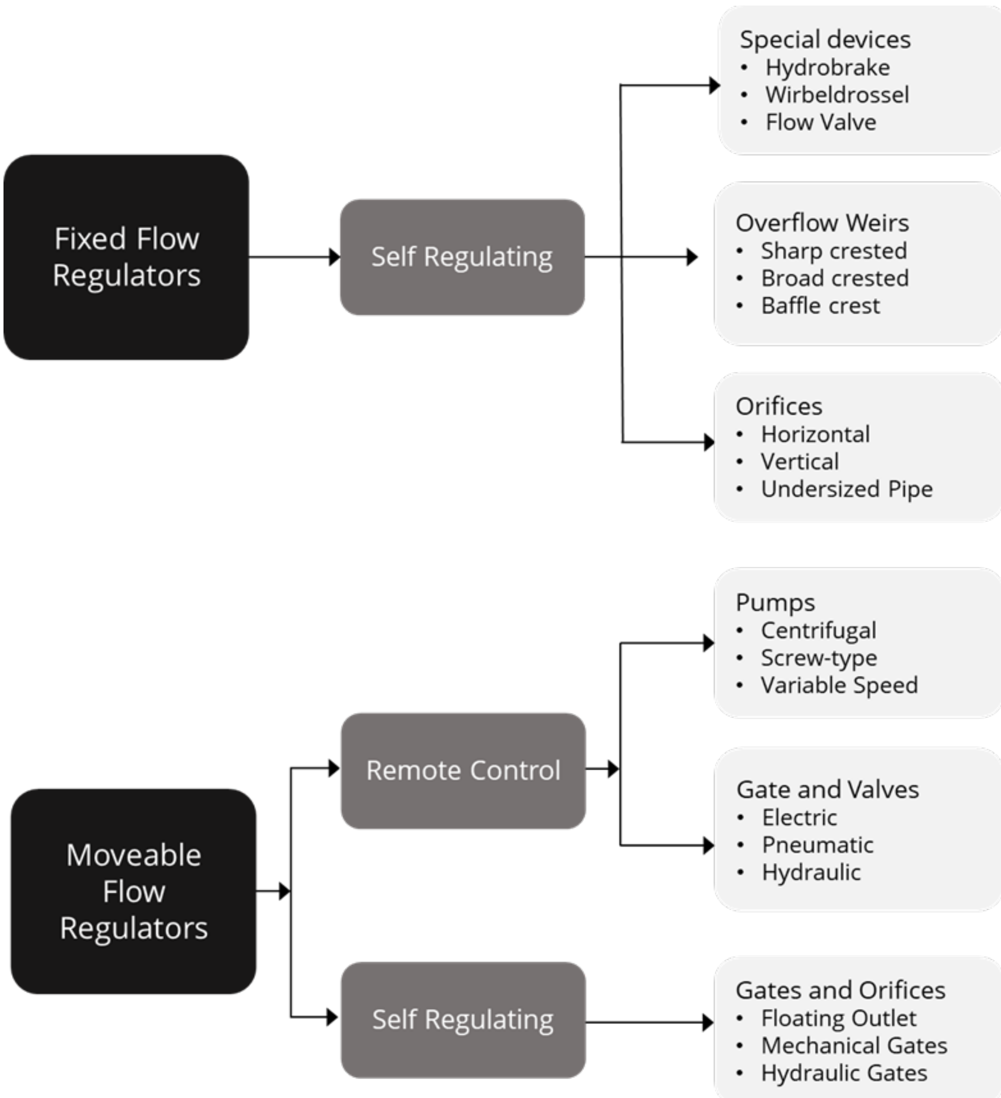


Figure 3.1: Various Types of Outflow Control Devices

3.3.6.1.1 Combined Sewer Overflow Control Methods

Source Controls (Best Management Practices)

- Porous pavements.
- Flow detention.
- Rooftop storage.
- Area drains and roof leader disconnection.
- Utilization of pervious areas for recharge.
- Snow removal and de-icing control.
- Commercial/Industrial runoff control.
- Sewer line flushing.
- Catch basin cleaning.
- Identifying and/or eliminating sewer system cross connections.

Collection System Controls

- Existing system management and in-system modifications.
- Complete or partial sewer separation.
- Infiltration/inflow control.
- Polymer injection.
- Regulating devices and backwater gates.
- Remote monitoring and real-time control.
- Flow diversion.

Storage

- In-system storage.
  - Inflatable dams.
  - Manual and automatic valves and gates.
- Surface storage.
- Off-line storage.
  - Storage tanks.
  - Lagoons.
  - Deep tunnels.
  - Abandoned pipelines.
  - In-receiving water flow balance method.
  - Street storage.

Physical Treatment

- Sedimentation.
- Dissolved Air Flotation (DAF).
- Screens.
  - Bar screens and coarse screens.
  - Fine screens and micro-strainers.
- Filtration.
- Flow concentrators.

Biological Treatment

- Activated sludge.
- Trickling filtration.
- Rotating Biological Contactors (RBCs).
- Treatment lagoons.
  - Oxidation ponds.
  - Aerated lagoons.
  - Facultative lagoons.
- Land treatment.

Physical-Chemical Treatment

- Chemical clarification.
- Filtration.
- Carbon absorption.
- High gradient magnetic separation.

Chemical Treatment (disinfection)

- Chemical.
- Radiation.

3.3.6.1.2 Treated for Combined Sewer Overflows

Treatment methods for CSOs can be classified as physical, biological, physical-chemical, and chemical.

Physical

Physical treatment alternatives include sedimentation, DAF, screening, and filtration. Physical treatment operations are usually flexible enough to be readily automated and can operate over a wide range of flows. They can also stand idle for long periods of time without affecting treatment efficiencies.

Solids separation devices such as swirl concentrators and vortex separators have been used in Europe and, to a lesser extent, in North America. These devices are small, compact solid separation units with no moving parts. Operation of vortex separators is based on the movement of particles within the unit. Water velocity moves the particles in a swirling action around the separator, additional flow currents move the particles down, and a sweeping action moves heavier particles across the sloping floor toward the central drain. During wet weather, the outflow from the unit is throttled, causing the unit to fill and to self-induce a swirling vortex-like flow regime. In the device secondary flow currents rapidly separate settleable grit and floatation matter. Concentrated foul matter is intercepted for treatment, while the cleaner, treated flow discharge to receiving waters. The device is intended to operate under extremely high flow regimes.

A device more recently developed and termed the Continuous Deflection Separator (CDS) differs from the more traditional vortex separator in that it utilizes a filtration mechanism for solids separation and does not rely on secondary flow currents induced by the vortex action.

Biological and Physical-Chemical Treatment

The use of biological and physical-chemical treatment processes for the treatment of combined wastewater has some serious limitations:

- The biomass used to assimilate the nutrients in the combined wastewater must be kept alive during dry weather, which can be difficult except at an existing treatment plant.
- Biological processes are subject to upset when subjected to erratic loading conditions.
- The land requirements for this type of plant can be excessive in an urban area.
- Operation and maintenance can be costly, and facilities require highly skilled operators.

It is feasible and frequent in practice, however, to treat a portion of the wet-weather flow at the treatment plant. In some treatment facilities the wet-weather flow receives full secondary treatment, whereas in others the flow is split, with some receiving primary treatment and disinfection only and the remainder receiving full secondary treatment.

Chemical Treatment (Disinfection)

Refer to Chapter 9 for disinfection requirements.

3.4 Details of Design & Construction

3.4.1 Sewer Capacity

Sewers should be designed to provide capacity for the peak anticipated wastewater flow and maximum I&I allowance without surcharge when flowing full.

3.4.2 Pressure Pipes

Sanitary sewers may be designed as pressure pipes provided that the hydraulic gradient for maximum flow is below basement elevations.

3.4.3 Minimum Pipe Size

Public sewers should not be less than 200 mm in diameter, however, under limited circumstances, such as effluent from Septic Tank Effluent Pump Systems (STEPS) and Septic Tank Effluent Gravity Systems (STEGS) may be allowed if the System Owner can demonstrate that the proposed sewer size is adequate and will not be detrimental to the O&M of the sewer system.

The hydraulic capacity of a gravity sewer should be based on consideration of factors such as projected in-service roughness coefficient, projected future connections during design life, slope, pipe material, actual in-service flows, and projected flows, including climate change considerations. In general, sewers larger than the minimum size required should be chosen so that the minimum velocity at the average flow is not less than 0.6 m/s for self-cleansing purposes, and the maximum velocity at the peak design flow is not greater than 3.0 m/s to minimize turbulence and erosion. Under exceptional circumstances, where velocities greater than 3.0 m/s are attained, provisions should be made to protect against displacement by erosion and impact.

3.4.4 Depth

In general, sewers should be deep enough to prevent freezing and to receive wastewater from most basements. Where possible, the peak hydraulic grade line of the sewer should be 300 mm below the underside of footings.

3.4.5 Slope

Sewers should be laid with a uniform slope between manholes except for alternate wastewater collection systems.

3.4.5.1 Minimum Slopes

All sewers should normally be designed and constructed to give mean velocities, when flowing full, of not less than 0.6 m/s or greater than 4.5 m/s based on *Kutter's* or *Manning's* formula using "n" value of 0.013. Use of other practical "n" values may be permitted by the Regulatory Authority if deemed justifiable. Velocities above 4.5 m/s may be permitted with high velocity protection. The minimum slopes which will provide a velocity of 0.6 m/s when sewers are flowing full are provided in Table 3.2.

Table 3.2: Minimum Slopes for Full Pipe Velocity of 0.6 m/s

Sewer Size (mm)	Minimum Slope in Metres (/100 m)
200	0.40
250	0.28
300	0.22
350	0.17
375	0.15
400	0.14
450	0.12
525	0.10
600	0.08
675	0.067
750	0.058
900	0.046

If possible, a minimum slope of 0.5% (0.5 m/100 m) should be utilized.

3.4.5.2 Increased Slopes

To achieve 0.6 m/s flow velocities in sewers which will flow less than 1/3 full, steeper slopes than given above must be used where conditions permit. For instance, the minimum slopes mentioned above would have to be doubled when depth of flow is only 1/5 full and quadrupled when depth of flow is only 1/10 full to achieve 0.6 m/s flow velocity.

3.4.5.3 Reduced Slopes

Under special conditions, if full and justifiable reasons are given, slopes slightly less than those required for the 0.6 m/s velocity when flowing full may be permitted. Such decreased slopes will only be considered where the depth of flow will be 30% of the diameter or greater for design average flow. Whenever such decreased slopes are selected, the Design Engineer must furnish with their report the computations of the anticipated flow velocities of average and daily or weekly peak flowrates. The pipe diameter and slopes should be selected to obtain the greatest practical velocities to minimize settling problems. The System Owner of the sewer system will give written assurance to the appropriate Regulatory Authority that any additional sewer maintenance required by reduced slopes will be provided.

3.4.5.4 High Velocity Protection

Where velocities greater than 4.5 m/s are unavoidable, special provisions should be made to protect against displacement by erosion and shock.

3.4.5.5 Steep Slope Protection

Sewers on 20% slopes or greater should be anchored securely with concrete anchors or equal, spaced as follows:

- Not over 11.0 m center to center on grades 20% and up to 35%.
- Not over 7.3 m center to center on grades 35% and up to 50%.
- Not over 5.0 m center to center on grades 50% and over.

3.4.6 Alignment

Sewers 600 mm or less in diameter should be laid with a straight alignment between manholes.

3.4.7 Curvilinear Sewers

Curvilinear sewers may be considered for pipe sizes more than 600 mm, with the following restrictions applicable:

- The sewer should be laid as a simple curve of a radius equal to or greater than 60 m.
- Manholes should be located at the ends of the curve and at intervals not greater than 90 m along the curve.
- The curve should run parallel to the curb or street centerline.
- The minimum grade on curved sewers should be 50% greater than the minimum grade required for straight runs of sewers. This requirement will be waived if the designer submits calculations to demonstrate that increased slope is not required to achieve self-cleansing velocity.
- Length of pipe should be such that deflections at each joint should be less than the allowable maximum recommended by the manufacturer.
- In general, curved sewers should be used only where savings in costs or the difficulty of avoiding other utilities necessitates their use.

3.4.8 Changes in Pipe Size

When a sewer joins a larger one at a manhole, the invert of the larger sewer should be lowered sufficiently to maintain the same energy gradient. An approximate method of securing these results is to place the 0.8 depth point of both sewers at the same elevations. Changes in size of sewers less than or equal to 600 mm should be at manholes only.

3.4.9 Allowance for Hydraulic Losses at Sewer Manholes

Differences in elevation across manholes should be provided to account for hydraulic losses. The elevation drop may be calculated using the head loss formula:

Head loss across manholes:

$$H = k\,(V_2^2 - V_1^2)/2g$$

Where:

H	=	Head loss	m
k	=	Coefficient	Dimensionless
V_1	=	Entrance velocity	m/s
V_2	=	Exit velocity	m/s
g	=	Acceleration due to gravity	m/s^2

Where sewer velocities are less than 2.5 m/s and the velocity change across the manhole is less than 0.6 m/s the invert drop may be determined using Table 3.3.

Table 3.3: Recommended Invert Drop

Invert Drop		
A	Straight run	15 mm
B	45° turn	30 mm
C	90° turn	60 mm

3.4.10 Sewer Services

Sewer services should be consistent with municipal or provincial requirements. It is required that unless "Tees" or "Wyes" have been installed, that saddles be used in connecting the service to the sewer. Generally, these are placed at an angle of 45° above horizontal. Connections should be made by authorized personnel only.

3.4.11 Sulphide Generation

Where sulphide generation in the wastewater collection system is a possibility, the problem should be minimized by designing sewers to maintain flows at a minimum cleansing velocity of 1.0 m/s.

3.4.12 Materials

Any accepted material for sewers will be given consideration, but the material selected should be adaptable to local conditions, such as character of industrial wastes, possibility of septicity, soil characteristics, exceptionally heavy external loading, abrasion, and similar problems.

All sewers should be designed to prevent damage from super-imposed loads. Proper allowance for loads on the sewer should be made because of the width and depth of the trench. When standard strength sewer pipe is not sufficient, the additional strength needed may be obtained by using extra strength pipe or by special construction.

3.4.13 Metering & Sampling

Where no other flow or sampling measuring devices are provided, one manhole on the outfall line should be constructed with a suitable removable weir for flow measurements and sampling. Easy access for flow measurement and sampling should be provided. Similar manholes should be constructed on sewer lines from industries to facilitate checking the volume and composition of the waste.

3.5 Installation

3.5.1 Standards

Installation specifications should contain appropriate requirements based on the criteria, standards, and requirements established by industry in its technical publications. Requirements should be set further in the specifications for the pipe and methods of bedding and backfilling thereof so as not to damage the pipe or its joints, impede cleaning operations and future tapping, nor create excessive side fill pressures or ovalization of the pipe, nor seriously impair flow capacity.

3.5.1.1 Trenching

The width of the trench should be ample to allow the pipe to be laid and jointed properly and to allow the backfill to be placed and compacted as needed. The trench sides should be kept as vertical as possible. When wider trenches are dug, appropriate bedding class and pipe strength should be used.

Ledge rock, boulders, and large stones should be removed to provide a minimum clearance of 150 mm below and on each side of all pipe(s).

3.5.1.2 Foundation

The foundation provides the base for the sewer pipe soil system. The project engineer should be concerned primarily with the presence of unsuitable soils such as peat or other highly organic or compressible soils, and with maintaining a stable trench bottom.

3.5.1.3 Bedding

Bedding material and methodology should be done in accordance with specifications from local jurisdiction and should be no less than as recommended by the pipe manufacture. Pipe and fittings must not be laid when the trench bottom is frozen, under water or when trench conditions or weather are unsuitable. Material removed from the trench should not be used as a bedding material.

3.5.1.4 Haunching

The material placed at the sides of a pipe from the bedding up to the spring line is the haunching.

Material used for sewer pipe haunching should be shovel sliced or otherwise placed to provide uniform support for the pipe barrel and to fill completely all voids under the pipe. Haunching material is to be compacted manually. The material used may be similar to the material used for bedding. Material removed from the trench should not be used as haunching material.

3.5.1.5 Initial Backfill

Initial backfill is the material which covers the sewer pipe and extends from the haunching to a minimum of 300 mm above the top of the pipe. Its function is to anchor the sewer pipe, protect the pipe from damage by subsequent backfill, and insure the uniform distribution of load over the top of the pipe. It should be placed in layers. The material used for initial backfill may be similar to the material used for bedding and haunching, however, it should be of a material which will develop a uniform and relatively high density with little compactive effort. Material removed from the trench should not be used as initial backfill.

3.5.1.6 Final Backfill

Final backfill is the material which extends from the top of the initial backfill to the top of the trench. It should be placed in 300 mm layers.

The material consists of the excavated material containing no organic matter or rocks having any dimension greater than 200 mm. In most cases, final backfill does not affect the pipe design. Compaction of the final backfill is usually controlled by the location as follows:

Table 3.4: Backfill Compaction

Traffic areas	95% of modified Proctor density required.
General urban areas	90% of modified Proctor density may be adequate.
Undeveloped areas	85% of modified Proctor density may be required.

Trench backfilling should be done in such a way as to prevent dropping of material directly on the top of pipe through any great vertical distance.

3.5.1.7 Borrow Materials

The material removed from the trench is not to be used as part of the bedding, haunching, nor initial backfill, therefore, material must be imported from another source. Borrow material must meet the specifications for final backfill.

Either cohesive or non-cohesive material may be used, however, the project engineer should assess the possible change in groundwater movement if cohesive material is used in rock or if non-cohesive material is used in impermeable soil.

3.5.1.8 Deflection Test

- Deflection tests should be performed on all flexible pipe. The test should be conducted after the final backfill has been in place at least 30 days to permit stabilization of the soil-pipe system.
- No pipe should exceed a deflection of 5%. If deflection exceeds 5%, replacement or correction should be accomplished in accordance with requirements in the approved specifications.
- The rigid ball, mandrel, or an approved electronic device used for the deflection test should have a diameter not less than 95% of the base inside diameter or average inside diameter of the pipe depending on which is specified in the applicable American Society for Testing and Materials (ASTM) specification, to which the pipe in manufactured. The test should be performed without mechanical pulling devices.

3.5.2 Joints

The installation of joints and the materials used should be included in the specifications. Sewer joints should be designed to minimize infiltration and to prevent the entrance of roots throughout the life of the system.

3.6 Sewer Rehabilitation Methods

3.6.1 Sewer Replacement

Sewer replacement is the most expensive method of sewer rehabilitation. In cases where there is evidence of structural damage or where differential settlement has altered the sewer grade, sewer replacement may be the only reasonable approach.

3.6.2 Sewer Relining

Sewer relining involves inserting a layer of piping material with a smaller diameter inside an existing pipe.

3.6.3 Lining & Slip Lining

Lining materials can range from cement applied directly to the inside of the existing pipe to modern plastics. Continuous plastic linings can reduce infiltration completely, though the net I&I control effectiveness of slip lining is a function of the integrity of sealing the annular space between the outside of the liner and the inside of the original pipe. Continuous grouting of the annular space will produce a more reliable seal than just packing the annular space at manhole pipe protrusions. The long-term integrity of High-Density Polyethylene (HDPE) has been shown, however, long–term net effectiveness will be more a function of the life of the annular space sealant.

Piping materials that are inserted but use the methods of joining pipe sections have a greater chance of leakage but still can be highly resistant to infiltration with effective annular space sealing and jointing technique. Where existing lateral to main line connections are sound, hook up of laterals is limited to cutting out the part of the lining covering the lateral and sealing the annular space. The integrity of this sealing step is a major factor in the overall infiltration reduction effectiveness. If the existing lateral to main line connection is not sound, a new lateral connection directly to the liner by a pipe saddle arrangement can achieve the best results. Typically, this will require external exposure of the lateral, requiring extreme care in the backfilling operation. Lining and sealing the annular space and careful lateral reconnections can be as effective in controlling I&I as replacement methods.

3.6.4 Cured-In-Place Pipe Lining

Due to the close contact with the inside of the original pipe, Cured-In-Place Pipe (CIPP) lining eliminates annular space leakage. Cured-in-place pipe lining can be installed using inversion lining or pull-in-place methods and can be hot water, steam, or Ultraviolet (UV) cured. If the part of the lining that covers the laterals is cut out properly, leakage around the laterals can be reduced to a low value. Lack of care in this step can result in poor infiltration control. Cured-in-place pipe lining can be effective in controlling I&I as a replacement method and does not require excavation to reconnect laterals if the existing lateral to main line hookup is in sound condition.

Cured-in-place pipe lining can be used for lining manholes and should exhibit the same high degree of infiltration reduction shown in sewer pipes. Openings to the sewers entering a manhole should be made carefully, as leakage could significantly reduce the overall effect of lining.

3.6.5 Sewer Sealing

Chemical grout sealers for internal grouting of small to medium sewers are widely accepted in the sewer maintenance industry, with even relatively small utilities owning their own grout packers and sealing equipment. The effectiveness of chemical grouting to seal a leaking joint is a function of the condition and structural stability of the pipe, the surrounding backfill material, and the quality of workmanship. Chemical grouting using conventional packing equipment is most effective where the failed element is the joint, not the pipe material.

Where grout is correctly applied, it is effective in preventing infiltration for a joint, however, the high degree of effectiveness only applies to the sealed joint, not necessarily to the section of pipe.

Leakage from service laterals, joints close to service laterals, adjacent pipe sections, and defects not correctable by the sealing procedure can render infiltration removal less effective.

3.6.6 Service Lateral Rehabilitation

Service laterals can constitute a serious source of I&I. The rehabilitation methods applied to the main sewer line including slip lining, CIPP lining, and grouting have been adapted for rehabilitating service laterals in addition to excavation and replacement.

In addition to I&I from the laterals, infiltration frequently results from leaky connection of the lateral to the main sewer and leakage at main sewer joints close to the lateral. Effective I&I control requires testing and repairing these sources of infiltration.

3.7 Inflow Control

Inflow is controlled by disconnecting the pathway by which storm-generated surface waters enter the sewer. Typical pathways are manhole covers, catch basins, area drains, and roof drain downspouts.

3.7.1 Manholes

Manhole covers containing vent and pick holes can allow significant sources of inflow when they are located in the path of surface runoff. Replacement with a waterproof, gasketed cover is estimated to be 90% effective in reducing inflow.

Manholes frequently leak between the frame and corbel, especially if there is heaving of the pavement from freezing. Use of elastomeric sealants poured or trowelled on the outside of the manhole or elastic sleeves is estimated to be 90% effective in reducing flow. Application of an adhesive sealant to the interior of the corbel and joint beneath the flange of the manhole frame is estimated to be only 75% effective because water can still enter the space between the frame and corbel, increasing the chance for seal failure from frost action.

3.7.2 Catch Basins

Catch basins and area drains connected to sanitary sewers can contribute large amounts of inflow. Plugging the connection to the sanitary system and reconnection to a storm drain is estimated to be 90% effective in reducing inflow. The effectiveness is estimated to be less than 100% to compensate for migration of some water to other parts of the sanitary sewer system.

3.7.3 Roof Drain

Downspouts or roof drains are frequent sources of inflow. Disconnection of these from sanitary sewer systems and reconnection to a storm sewer is estimated to be 90% effective in reducing inflow, with the remaining 10% finding its way to the sewer system by other routes. Where the disconnected downspout is discharged on the ground surface rather than being connected to a storm sewer, the inflow reduction is likely to be significantly less (possibly zero if service laterals serving the property are in poor condition).

3.7.4 Other

Sump pump and foundation drain connections to sanitary sewers represent other significant sources of inflow. Disconnection of these sources and reconnection to storm sewers results in approximately 75% inflow reduction. Any discharge of these disconnected sources to the ground surface prevents net reduction. To maintain long-term effective control requires an effective enforcement program to preclude reconnection.

3.8 Trenchless Construction

Traditional construction methods for the installation, replacement, or rehabilitation of underground utilities such as watermains and sewers require extensive excavation and surface disruption. Trenchless technologies is an umbrella term for a variety of construction techniques and methods that significantly reduce excavation and surface disruption. Trenchless technologies offer the ability to install, replace, or rehabilitate buried infrastructure while reducing the disruption to the surrounding community, environment, and other utilities located in the vicinity.

There are varying levels of structural enhancement that can be provided by trenchless rehabilitation techniques; from non-structural linings to fully structural linings. Non-structural enhancements are typically used to stop leaks (e.g., I&I), enhance system hydraulics or water quality, or provide corrosion protection. Fully structural linings, on the other hand, provide a complete replacement to the host pipe system upon completion.

Trenchless technologies are particularly popular in urban areas where there is a high density of underground utilities and/or vehicular and pedestrian traffic. Trenchless technologies also offer advantages for crossing transportation corridors, waterways, and environmentally sensitive areas.

The trenchless technology industry is an advancing field with new or modified methods for trenchless construction continually under development. Some of the most common trenchless technologies are listed as follows:

Trenchless installation methods:

- Tunnelling.
- Directional drilling.
- Pipe ramming.
- Pipe jacking.
- Auger boring.

Trenchless replacement/rehabilitation methods:

- Slip lining.
- Pipe bursting.
- Cured in place pipe liners.
- Sprayed in Place Pipe (SIPP) liners.
- Grouting.

As the technologies available are constantly advancing and changing, the list above simply provides a sample of the commonly used practices. The use of trenchless technologies should be considered in comparison to traditional open-cut methods for the replacement, installation and/or rehabilitation of underground utilities on a case-by-case basis.

Further information can be found in the *M28 Rehabilitation of Water* from the American Water Works Association.

3.9 Manholes

3.9.1 Location

Manholes should be located at all:

- Junctions.
- Changes in grade.
- Changes in size.
- Change in alignment (except with curvilinear sewers).
- Termination points of sewers.

3.9.2 Spacing

The maximum acceptable spacing for manholes is 120 m for sewers 400 mm in diameter or less. Spacing of up to 150 m may be used for sewers 450 mm to 750 mm in diameter. Spacing of up to 180 m may be considered in cases where cleaning equipment is available and capable of maintaining the collection system. Larger sewers may use greater manhole spacing.

Cleanouts may be used only with approval of the System Owner and Regulatory Authorities and should not be substituted for manholes nor installed at the end of laterals greater than 45 m in length.

3.9.3 Minimum Diameter

The minimum diameter of a sanitary manhole should be 1,050 mm.

3.9.4 Drop Manholes

A drop pipe should be provided for a sewer entering a manhole at an elevation of 600 mm or more above the manhole invert. Where the difference in elevation between the incoming sewer and the manhole invert is less than 600 mm the invert should be filleted to prevent solids deposition.

Inside drop connections (when necessary) should be secured to the interior wall of the manhole and provide access for cleaning.

Due to the unequal earth pressures that would result from the backfilling operation in the vicinity of the manhole, the entire outside drop connection should be encased in low-strength concrete.

3.9.5 Pipe Connections

A flexible watertight joint should be provided on all pipes, within 300 mm of the outside wall of the manhole.

3.9.6 Frost Lugs

Where required, frost lugs should be provided to hold precast manhole sections together.

3.9.7 Frame & Cover

The manhole frame and cover should be made of cast iron and designed to meet the following conditions:

- Adequate strength to support superimposed loads.
- Provision of a good fit between cover and frame to eliminate movement in traffic.
- A reasonably tight closure.

3.9.8 Watertightness

Manholes should be of the pre-cast or poured-in-place concrete type, or of another type approved by the Regulatory Authorities. All manhole joints must be watertight, and the manhole should be waterproofed on the exterior, if required.

Watertight manhole covers are to be used wherever the manhole tops may be flooded by street runoff or high-water level. Consideration should be given where manhole structures and covers may be more prone to flooded conditions including climate change considerations. Locked manhole covers may be desirable in isolated easement locations, or where vandalism may be a problem.

3.9.9 Flow Channel & Benching

The channel should be, as far as possible, a smooth continuation of the pipe. The completed channel should be U-shaped.

3.9.9.1 Small Pipe Channel

For sewer sizes less than 375 mm, the channel height should be at least 1/2 the pipe diameter.

3.9.9.2 Large Pipe Channel

For sewer sizes 375 mm and larger, the channel height should not be less than 3/4 of the pipe diameter.

3.9.9.3 Bench Area

The bench should provide good footing for a worker and a place for tools and equipment.

3.9.9.4 Bench Slope

Benching should be at a slope of at least 1-to-12 (vertical-to-horizontal) and not greater than 1-to-8. Benching should have a wood float finish.

3.9.10 Corrosion Protection

Where corrosion is anticipated because of either sulphate attack or sulphides, consideration should be given to the provision of corrosion resistant material or effective protective linings.

3.10 Testing & Inspection

3.10.1 General

Each section of a sanitary sewer should be tested for exfiltration and/or infiltration. A section is the length of pipe between successive manholes or termination points, including service connections.

Each section of a sewer, and its related appurtenances, should be flushed prior to testing. The method of testing should be as described in the construction specifications. In the absence of such specifications the following testing method will apply.

3.10.2 Exfiltration Test

Each sewer section should be filled with water and a nominal head should remain on the section for 24 hours immediately prior to testing.

Water should be added to the section to establish a test head of 1.0 m over either the crown of the pipe, measured at the highest point of the section, or the level of static groundwater, whichever is greater. This may be increased by the inspector to satisfy local conditions.

The test head should be maintained for 1 hour. The volume of water required to maintain the head during the test period should be recorded.

3.10.3 Infiltration Test

Infiltration tests should be conducted in lieu of exfiltration tests where the level of static groundwater is 750 mm or more above the crown of the pipe, measured at the highest point in the section.

A 90° V-notch weir should be placed in the invert of the pipe at the downstream end of the section. The total volume of flow over the weir for 1 hour should be measured and recorded.

3.10.4 Allowable Leakage

Allowable leakage should be determined by the following formula:

$$L = F \times D \times \frac{S}{100}$$

Where:

L	=	Allowable leakage (in L/hour).	
D	=	Diameter (in mm).	
S	=	Length of section (in m).	
		Leakage factor (L/hour per mm of diameter per 100 m of sewer):	
		Exfiltration test :	
		Porous pipe	F = 0.12 L
F	=	Non-porous pipe	F = 0.02 L
		Infiltration test :	
		Porous pipe	F = 0.10 L
		Non-porous pipe	F = 0.02 L

3.10.5 Low Pressure Air Testing

Air testing equipment should be designed to operate above ground. No personnel will be permitted in the trench during testing. Air testing will not be permitted on pipes with diameter greater than 600 mm.

The test section should be filled with air until a constant pressure of 28 kPa (4 psi) is reached and allowed to stabilize for at least five minutes. After the 5-minute period, the air supply should be shut off, and the pressure decreased to 24 kPa (3.5 psi). The time required for the pressure to reach 17 kPa (2.5 psi) should be measured.

3.10.6 Allowable Time for Air Pressure Decrease

Minimum times allowed for air pressure drop are provided in Table 3.5.

Table 3.5: Minimum Specified Time Required for Air Testing

Minimum Specified Time Required for Air Testing											
Pipe Dia. (mm)	Min. Time (min: sec)	Length for Min. Time (m)	Time for Longer Length (sec)	Specification Time for Length (L) Shown (min:sec)							
				30 m	45 m	60 m	75 m	90 m	105 m	120 m	135 m
100	1:53	182	0.62 L	1:53	1:53	1:53	1:53	1:53	1:53	1:53	1:53
150	2:50	121	1.40 L	2:50	2:50	2:50	2:50	2:50	2:50	2:50	3:10
200	3:47	91	2.49 L	3:47	3:47	3:47	3:47	3:47	4:22	4:59	5:37
250	4:43	73	3.89 L	4:43	4:43	4:43	4:51	5:49	6:47	7:46	8:44
300	5:40	61	5.61 L	5:40	5:40	5:40	6:58	8:23	9:47	11:11	12:35
375	7:05	48	8.76 L	7:05	7:05	8:50	11:02	13:13	15:24	17:36	19:47
450	8:30	41	12.60 L	8:30	9:20	12:29	15:38	18:47	21:56	25:05	28:14
525	9:55	35	17.20 L	9:55	13:05	17:27	21:49	26:11	30:32	34:54	39:16
600	11:20	30	22.40 L	11:24	17:57	22:48	28:30	34:11	39:53	45:35	51:17

3.10.7 Sewer Inspection

Manholes and sewers should be inspected for watertightness, prior to placing into service.

3.10.7.1 Video Inspection

Inspection of 100% of the sewer using the Closed-Circuit Television (CCTV) method and recorded on videotape should be specified. This should be conducted within the 1-year guarantee period. This inspection should be carried out preferably during the periods of high groundwater table in the spring or fall, or at the discretion of the System Owner.

3.10.7.2 Inspection Record

The complete record of the inspection should be the property of the System Owner. The original video and one edited copy of the video of the sections showing defects should be turned over to the System Owner.

3.10.7.3 Record Content

The maximum speed of the television camera through the pipe should be 0.30 m/s with a 5 second minimum stop at each defective location and a 15 second minimum stop at each lateral showing a flow discharging into the pipe. The video should display distances at a maximum interval of 3.0 m and a brief description of every defective location and of each service connection.

3.11 Inverted Siphons

Inverted siphons should have not less than two barrels with a minimum pipe size of 150 mm and should be provided with necessary appurtenances for convenient flushing and maintenance. The manholes should have adequate clearances for rodding; and in general, sufficient head should be provided and pipe sizes selected to secure velocities of at least 0.9 m/s for average flows. The inlet and outlet details should be so arranged that the normal flow is diverted to one barrel and that either barrel may be cut out of service for cleaning. The vertical alignment should permit cleaning and maintenance.

3.12 Protection of Water Supplies

3.12.1 Water-Sewer Cross Connections

There should be no physical connection between a public or private potable water supply system and a sewer, or appurtenance thereto, which would permit the passage of any wastewater or non-potable water into the potable supply. No water pipe should pass through or come into contact with any part of a sewer manhole, gravity sewer, or wastewater force main.

3.12.2 Relation to Water works Structures

While no general statement can be made to cover all conditions, it is recognized sewers should be kept remote from public water supply wells or other water supply sources and structures.

3.12.3 Relation to Water Mains

3.12.3.1 Horizontal & Vertical Separation

Whenever possible, sewers should be laid at least 3.0 m horizontally, from any existing or proposed water main. Should local conditions prevent a lateral separation of 3.0 m, a sewer may be laid closer than 3.0 m to a water main if:

- It is laid in a separate trench.
- It is laid in the same trench, with the water main located at one side with a minimum horizontal separation of 300 mm and on a bench of undisturbed earth.

If in either case the elevation of the top (crown) of the sewer is at least 300 mm below the bottom (invert) of the water main or as required by the Regulatory Authority having jurisdiction. Where a water main must be installed paralleling a gravity sewer and at a lower elevation than the gravity sewer, the water main must be installed in a separate trench. The soil between the trenches must be undisturbed.

3.12.3.2 Crossings

Whenever sewers must cross under the water mains, the sewer should be laid at such an elevation that the top of the sewer is at least 450 mm below the bottom of the water main. When the elevation of the sewer cannot be varied to meet the above requirement, the water main should be relocated to provide this separation or reconstructed with mechanical joint pipe for a distance of 3.0 m on each side of the sewer. One full length of water main should be centred over the sewer so that both joints will be as far from the sewer as possible.

3.12.3.3 Special Conditions

When it is impossible to obtain proper horizontal and vertical separation as stipulated above, the sewer should be designed and constructed equal to water pipe and should be pressure-tested to assure water-tightness.

3.12.3.4 Warning/Marker & Detection Tape

Warning/marker and detection tape should be installed continuously with a minimum 1.0 m overlap at joints above water, sewer, and force mains. Warning/marker tape should be heavy gauge polyethylene, 150 mm wide and indicate the service line below. Detectable tape should be either fabricated of detectable metallic material for underground installation or corrosion resistant insulated wires embedded in warning/marker tape. Detection tapes are intended for pipe location and must be installed above the pipe at an elevation of 300 mm below ground surface and be detectable using conventional pipe location apparatus.

3.13 Sewers in Relation to Watercourses

3.13.1 Location of Sewers in Relation to Watercourses

3.13.1.1 Cover Depth

The top of all sewers entering or crossing watercourses should be at a sufficient depth below the natural bottom of the stream bed to protect the sewer line. In general, the following cover requirements must be met:

- A cover of 0.3 m is required where the sewer is located in rock.
- A cover of 0.9 m is required in other material. In major streams, more than 0.9 m of cover may be required
- In paved stream channels, the top of the sewer line should be placed below the bottom of the channel pavement.

More cover than stated above may be required. Site-specific conditions must be evaluated by the designer to determine necessary design elements. Less cover will be approved only if the proposed sewer crossing will not interfere with the future improvements to the stream channel. Reasons for requesting less cover should be given in the project proposal.

3.13.1.2 Horizontal Location

Sewers located along watercourses should be located outside of the stream bed and with sufficient setback to provide for future possible stream widening and potential changes to flood extent including climate change considerations, and to prevent pollution by siltation during construction.

3.13.1.3 Structures

The sewer outfalls, headwalls, manholes, gate boxes, or other structures should be located so they do not interfere with the free discharge of flood flows of the watercourse. The calculation of flood flows should include climate change considerations (e.g., increases in the intensity of precipitation events and sea level rise).

3.13.1.4 Alignment
Sewers crossing watercourses should be designed to cross the watercourse as nearly perpendicular to the stream flow as possible and should be free from change in grade. Sewer systems should be designed to minimize the number of stream crossings.

3.13.2 Construction

3.13.2.1 Materials
Sewers entering or crossing watercourses should be constructed of cast or Ductile Iron (DI) pipe with mechanical joints. Otherwise, they should be constructed so they will remain watertight and free from changes in alignment or grade. Material used to backfill the trench should be stone, coarse aggregate, washed gravel, or other materials which will not cause siltation.

3.13.2.2 Siltation & Erosion
Construction methods that will minimize siltation and erosion should be employed. The Design Engineer should include in the project specifications the method(s) to be employed in the construction of sewers in or near watercourses to provide adequate control of siltation and erosion. Specifications should require that cleanup, grading, seeding, and planting or restoration of all work areas should begin immediately. Exposed areas should not remain unprotected for more than 7 days.

3.14 Aerial Crossings
Support should be provided for all joints in pipes utilized for aerial crossings. The supports should be designed to prevent frost heave, overturning, and settlement.

Precautions against freezing such as insulation and increased slopes should be provided. Expansion jointing should be provided between above-ground and below-ground sewers. Precautions should include climate change considerations such as potential changes to freeze-thaw cycles.

For aerial watercourse crossings, the impact of flood waters debris and climate change impact should be considered. The bottom of the pipe should be placed no lower than the elevation of the 1-in-100-year flood level including climate change considerations for the peak flood elevation.

3.15 Alternative Wastewater Collection Systems
Alternative wastewater collection systems are options to be considered for servicing small existing or new rural developments when the estimated costs of a conventional wastewater collection system is prohibitive. Two options are generally considered:

- Septic tank effluent gravity systems.
- Small Diameter Pressure Sewers (SDPS).

3.15.1 Septic Tank Effluent Gravity Systems
Septic tank effluent gravity systems require preliminary treatment through the use of septic tanks upstream of each connection. With the solids removed, the collector mains need not be designed to carry solids as conventional sewers must be. Collector mains are smaller in diameter and laid with variable or inflective

gradients. The required size and shape of the mains is dictated primarily by hydraulics rather than solids carrying capabilities.

3.15.1.1 House Connections

House connections are made at the inlet to the septic tank. All household wastewaters enter the system at this point.

3.15.1.2 Septic Tanks

Septic tanks are underground, watertight tanks with baffled inlets and filtered outlets. They are designed to remove both floating and settleable solids from the waste stream through quiescent settling. Ample volume is provided for storage of the solids which must be periodically removed through an access port. Typically, a single-chamber septic tank, vented through the house plumbing stack vent, is used.

Grease tanks are used to intercept most greases and solids before they enter a wastewater disposal system. Common wastewater contains small amounts of oils which enter into septic tanks and treatment facilities to form a floating scum layer. Grease tanks should be considered for commercial industrial facilities with potential of discharging fats, oils, and greases. Tanks should be sized by engineer, complete with filters, alarms and include a discharge a minimum of 300 mm off the bottom of the tank.

3.15.1.3 Service Laterals

Service laterals connect the septic tank with the collector main. Typically, they are 75 to 100 mm in diameter, but should be no larger than the collector main to which they are connected. They may include a check valve or other backflow prevention device near the connection to the main.

3.15.1.4 Collector Mains

Collector mains are small diameter plastic pipes with typical minimum diameters of 75 to 100 mm. The mains are trenched into the ground at a depth sufficient for frost protection and to collect the septic tank effluent from most connections by gravity. Unlike conventional gravity sewers, STEGS are not necessarily laid on a uniform gradient with straight alignments between cleanouts or manholes. In places, the mains may be depressed below the hydraulic grade line. Also, the alignment may be curvilinear between cleanouts to avoid obstacles in the path of sewers.

3.15.1.5 Cleanouts & Vents

Cleanouts and vents provide access to the collector mains for inspection and maintenance. Cleanouts are preferable to manholes because they are less costly and can be more tightly sealed to eliminate most infiltration and grit which commonly enter through manholes. Vents are necessary to maintain free flowing conditions in the mains. Vents in household plumbing are sufficient except where depressed sewer sections exist. In such cases, air release valves or ventilated cleanouts may be necessary at the high points of the main.

Manholes should only be used where cleanouts are not feasible. As discussed above, manholes, when used, must be designed to prevent I&I through waterproofing, sealed lids, etc.

3.15.1.6 Lift Stations

Lift stations are necessary where the elevation differences do not permit gravity flow. Either STEPS or mainline lift stations may be used. Septic tank effluent gravity systems units are small lift stations installed to pump wastewater from one or a small cluster of connections to the collector main, while a mainline lift station is used to service all connections in a larger drainage basin.

3.15.1.7 Design Criteria

Peak flows are based on the following formula:

$$Q = 1.262 + 0.032D$$

Where:

Q = Flow in L/second.
D = Number of equivalent dwelling units served.

*Above equation for 1.26 L/s (20 usgpm) pump.

A determination of peak flows is used for design instead of actual flow data. Each segment of sewer is analyzed by the Hazen-Williams or Manning equation. Roughness coefficients of 130 to 140 for Hazen-Williams and 0.011 for Manning's are commonly used. No minimum velocity is required. Check valves may be used in flooded or other sections on service laterals where backup from the main is possible.

All components must be corrosion-resistant and all discharges (e.g., to a conventional gravity interception or treatment plant) must be made through drop inlets below the liquid level to minimize odours. The system is ventilated through service-connection house vent stacks. Other atmospheric openings should be directed to sand beds for odour control unless they are located away from the populace.

Mainline cleanouts are spaced at 120 to 300 m apart. The septic (interceptor) tank effluent is assumed to contain 100 to 150 mg/L 5-Day Biochemical Oxygen Demand (BOD_5) and 50 to 75 mg/L Suspended Solids (SS).

3.15.1.8 Monitoring

Some management schemes involve biannual tank inspection, effluent filter cleaning, and pumping schedule (e.g., 3 to 5 years for residential users and every year for commercial users). For monitoring requirements, refer to local jurisdiction.

3.15.2 Small Diameter Pressure Sewers

Small diameter pressure sewers are small diameter pipelines, buried just below frost level, which follow the profile of the ground. Main diameters typically range from 50 to 150 mm with service lateral diameters of 25 to 38 mm. Polyvinyl Chloride (PVC) is the most common piping material. Piping should be pressure rated for the anticipated operating conditions.

Each home connected to the pipeline requires either a Grinder Pump (GP) or a STEPS. The major difference between the two pressure systems is in the on-site equipment and layout, as outlined in the following section. Modification of household pumping is not required for either system. Pressure systems do not have the large excess capacity typical of conventional gravity sewers, therefore, they must be designed with a balanced approach with consideration of future growth and internal hydraulic performance.

Grinder pump effluent is generally about twice the strength of the conventional sewer wastewater (i.e., Biochemical Oxygen Demand (BOD) and Total Suspended Solids (TSS)) of 350 mg/L. Septic tank effluent pump systems effluent is pre-treated and has a BOD_5 of 100 to 150 mg/L and SS of 50 to 70 mg/L.

3.15.2.1 Grinder Pump System

A GP pressure sewer has a pump and electrical service at each service connection. The pumps discharge building wastewater into a pressurized pipe system that terminates at a treatment plant or gravity collector. Since the mains are pressurized, there is no infiltration into them, however, I&I can occur in the house sewers and the pump wells.

The pipe network typically does not have closed loops. The sewer profile and the ground surface profile are often parallel, and the horizontal alignment can be curvilinear. Cleanouts are used to provide access for flushing. Automatic air release valves are required at and slightly downstream of summits in the sewer profile. Due to the small diameters and curvilinear horizontal and vertical alignment, excavation depths and volumes are typically smaller than conventional sewers, sometimes requiring only a chain trencher.

Grinder pump systems can use either centrifugal or positive displacement pumps. The choice is typically up to the Design Engineer. The positive displacement pumps have a discharge nearly independent of head, which may simplify some design problems, however, it may cause some additional operational problems.

3.15.2.2 Septic Tank Effluent Pump System

A STEPS system typically has a septic tank and a pump at each service connection. Electrical service is required at each service connection. The pumps discharge septic tank effluent into a pressurized pipe system that terminates at a treatment plant or a gravity sewer. Since the pipes are pressurized, there will be no inflow into them, but I&I into the house sewers and the septic/interceptor tanks should be minimized during construction of onsite facilities. The tanks remove grit, settleable solids, and grease. The discharge line from the pump is equipped with at least one check valve and one gate valve. The pipe network can contain closed loops but typically does not. The sewer profile and the ground surface profile are often parallel, and the horizontal alignment can be curvilinear. Cleanouts are used to provide access for flushing. Automatic air release valves are required at and slightly downstream of summits in all pressure sewer profiles. Considering the small diameter, curvilinear horizontal and vertical alignments, excavation depths, and volumes are typically much smaller for pressure sewers than for conventional sewers, sometimes requiring only a chain trencher for excavation.

3.15.2.3 Design Criteria

When positive displacement GP systems are used, the design flow can be obtained by multiplying the pump discharge by the maximum number of pumps expected to be operating simultaneously.

The following equation is used for centrifugal pumps:

$$Q = 1.262 + 0.032D$$

Where:

Q = Flow in L/second.

D = Number of equivalent dwelling units served.

*Above equation for 1.26 L/s (20 usgpm) pump.

The operation of the system under various assumed conditions should be simulated by a computer as a check on the adequacy of the design. Allowances for I&I are not required. No minimum velocity is used in design, but GP systems must attain 1.0 to 1.5 m/s at least once per day.

A Hazen-Williams coefficient (C) of 130 to 150 is suggested for hydraulic analysis.

Pressure mains generally use 50 mm or large PVC pipe, although 750 mm pipe is preferred owing to the availability of standard tapping equipment. Rubber-ring joints are preferred over solvent welding due to the high coefficient of expansion for PVC pipe. High-density polyethylene pipe with fused joints can also be used.

3.15.2.4 Monitoring

Detailed records of daily maintenance and annual summaries should be provided. Specific records for each unit should be kept with the community plan to permit maintenance staff to evaluate potential problems prior to the arrival at the site of the emergency call. On larger flow sources, cycle counters may be useful to track any trends, just as periodic line-pressure checks can alert the O&M staff to impending needs.

3.15.2.5 System Layout

Pressure sewer systems should be designed taking the following into consideration:

- Branched layout rather than looped.
- Maintain cleansing velocities especially when GP type pressure sewers are used.
- Minimize high head pumping and downhill flow conditions.
- Locate on lot facilities close to the home for ease of maintenance.
- Provide for each home to have its own pump chamber.
- Septic tanks are also needed with a STEPS system.

3.15.2.5.1 Other General Considerations

Following are general considerations when evaluating the small diameter servicing option, some of which have been mentioned previously.

3.15.2.5.2 Population Density

When housing is sparse, resulting in long reaches between services, the cost of providing conventional sewers is often prohibitive. Pressure sewers and STEGS are typically less costly on a lineal foot basis, so they often prove to be more cost-effective when serving sparse populations.

3.15.2.5.3 Ground Slopes

Where the ground profile over the sewer main slopes continuously downward in the direction of flow, conventional or STEGS are normally preferred. If intermittent rises in the profile occur, conventional sewers may become cost prohibitive. The variable grade gravity sewer variation of STEGS, by use of inflective gradients and in conjunction with STEPS pressure sewer connections, can be economically applied. Vacuum sewers may be particularly adaptable to this topographic condition, so long as head requirements are within the limits of available vacuum.

In flat terrain, conventional sewers become deep due to the continuous downward slope of the main, requiring frequent use of lift stations. Both the deep excavation and the lift stations are expensive. Septic tank effluent gravity systems are buried shallower, owing to the flatter gradients permitted. Pressure sewers are often found to be practical in flat areas, as ground slope is of little concern. In areas where the treatment plant or interceptor sewer are higher than the service population, pressure sewers are preferred, but should be evaluated against STEGs systems with lift stations.

3.15.2.5.4 Subsurface Obstacles

Where rock excavation is encountered, the shallow burial depth of alternative sewer mains reduces the amount of rock to be excavated.

Deep excavations required of conventional sewers sometimes encounter groundwater. Depending on severity, dewatering can be expensive and difficult to accomplish.

3.15.3 Detailed Design Guidelines

The above are general design considerations only.

3.16 References

Alberta Environment, *Guidelines for Municipal Wastewater Irrigation*, 2000.

Alberta Environment, *Standards and Guidelines for Municipal Waterworks, Wastewater, and Storm Drainage Systems*, January 2006. <https://open.alberta.ca/dataset/3f9fd1e5-34a8-4aa3-bbf0-7d6da6b087ab/resource/8ba41d4e-9b89-4807-b7f5-427edf85d28c/download/aenv-standards-and-guidelines-for-municipal-waterworks-wastewater-storm-drainage-systems-6979.pdf>.

ASTM International. *Standard Specification for Concrete Aggregates* - Designation C33/C33M – 18 gradation 67 (19-9.8 mm). ASTM, 2018 <https://www.astm.org/c0033_c0033m-18.html>.

Bowker, P.G.; Frigon, G.; Kreissl, J.F.; Otis, R.J. *EPA/625/R-92/005 Manual: Wastewater Treatment Disposal for Small Communities*. Cincinnati: U.S. Environmental Protection Agency, 1992. <https://cfpub.epa.gov/si/si_public_record_Report.cfm?Lab=NRMRL&dirEntryId=124773>.

British Columbia Ministry of the Environment, *Pollution Control Objectives for Municipal Type Waste Discharges in British Columbia*, 1989. <https://a100.gov.bc.ca/pub/eirs/finishDownloadDocument.do?subdocumentId=5563>.

Canadian Council of the Ministers of the Environment, *Canada-wide Strategy for the Management of Municipal Wastewater Effluent*. Canadian Council of the Ministers of the Environment, February 2009. <https://www.ccme.ca/en/res/mwwe_strategy_e.pdf>

Canadian Council of the Ministers of the Environment, *Technical Supplement 2. Canada-wide Strategy for the Management of Municipal Wastewater Effluent. Environmental Risk Management: Framework and Guidance*. Canadian Council of Ministers of the Environment, June 2008. <https://ccme.ca/en/res/mwwe_techsupp_2_e.pdf>.

Canadian Institute on Pollution Control, *Manual on Wastewater Sampling Practice*, 1972.

Government of Canada Justice Laws. *Fisheries Act*, RSC 1985, c F-14. Government of Canada, 1985. https://laws.justice.gc.ca/eng/acts/f-14/page-1.html

Mays, Larry W. *Stormwater Collection Systems Design Handbook*. McGraw-Hill Professional, 17 May 2001. <https://books.google.ca/books/about/Stormwater_Collection_Systems_Design_Han.html?id=Bki4_m-N_rAC&redir_esc=y>.

New Brunswick Department of Municipal Affairs and Environment, *Guidelines for the Collection and Treatment of Wastewater*, September 1987.

Ontario Ministry of the Environment, *Design Guidelines for Sewage Works*, May 2019. <https://www.ontario.ca/document/design-guidelines-sewage-works-0>.

Standards Council of Canada. *CSA W204:19 Flood Resilient Design of New Residential Communities*. Toronto: CSA Group, December 2019. <https://www.csagroup.org/store/product/CSA%20W204%3A19/>.

Tchobanoglous, George, Burton, Franklin L., Stensel, H. David, and Metcalf & Eddy. *Wastewater Engineering: Treatment and Reuse*. 4th Ed. / Revised by George Tchobanoglous, Franklin L. Burton, H. David Stensel. ed. McGraw-Hill Series in Civil and Environmental Engineering. Boston; Montreal: McGraw-Hill, 2003.

U.S. Environmental Protection Agency. EPA/625/1-91/024 *Alternative Wastewater Collection Systems*. USA, October 1991. <https://nepis.epa.gov/Exe/ZyNET.exe/300046DW.TXT?ZyActionD=ZyDocument&Client=EPA&Index=1991+Thru+1994&Docs=&Query=&Time=&EndTime=&SearchMethod=1&TocRestrict=n&Toc=&TocEntry=&QField=&QFieldYear=&QFieldMonth=&QFieldDay=&IntQFieldOp=0&ExtQFieldOp=0&XmlQuery=>.

U.S. Environmental Protection Agency. *Manual: Wastewater Treatment/Disposal for Small Communities*, EPA-625/1-92/005. Cincinnati, OH: Office of Research and Development, 1992.

Water Environment Federation, *Manual of Practice FD-6, Existing Sewer Evaluation & Rehabilitation*, 1994.

Water Environment Federation. *Alternative Sewer Systems FD-12*. 2nd ed. McGraw-Hill's Access Engineering. New York: McGraw-Hill, 2008.

Water Pollution Control Federation, *Manual of Practice FD-4 - Design of Wastewater and Stormwater Pumping Stations*, 1981.

Table of Contents

Chapter 4 Wastewater Pumping Stations

The latest edition of *CSA W204:19 Flood Resilient Design of New Residential Communities* should be referenced when designing wastewater pumping stations. Wastewater pumping stations have special risks and considerations pertaining to potential explosive conditions, corrosion, and personal safety. Careful attention and understanding of the latest edition of *NFPA 20 Standard for the Installation of Stationary Pumps for Fire Protection* and *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations* from the National Fire Protection Association (NFPA) is required.

4.1 General

Wastewater pumping station structures and electrical and mechanical equipment should be protected from physical damage from the 1-in-100-year flood including climate change considerations. Refer to Chapter 2 for further guidance.

The pumping station should be in an area where it will remain fully operational and fully accessible during flooding events, through the provision of back-up generators or alternative power supply.

The pumping station should be elevated above the 1-in-100-year flood elevation including climate change considerations. Refer to Chapter 2 for further guidance.

The pumping station should be protected to prevent vandalism and entrance by animals or unauthorized persons.

The pumping station should be located off street right-of-way in an appropriate area designated for pumping station purposes.

During preliminary location planning, consideration should be given to the potential of emergency overflow provisions and as much as possible the avoidance of health hazards, nuisances, and adverse environmental effects.

4.1.1 Design Capacity

4.1.1.1 Separate Sewer Systems

Pumps and controls should be able to pump the expected 25-year peak wastewater flows, under normal growth conditions, with the largest capacity pump out of operation. Wastewater pumping station facilities should be designed to accommodate the expected 25-year peak wastewater flows by upgrading pumps and controls. See Section 3.3 for the recommended approach for the calculation of peak wastewater flows. In certain cases, where it can be shown that staging of construction will be economically advantageous, lesser design periods may be used provided it can be demonstrated that the required capacity can be “online” when needed. Pumping station overflows should be permitted as outlined in Section 4.3.

If only two pumps are provided, they should have the same capacity. Each should be capable of handling the expected peak wastewater flow. Where three or more units are provided, they should be designed to fit actual flow conditions and must be of such capacity that with any one unit out of service the remaining units will have capacity to handle maximum wastewater flows, considering head losses associated with parallel operation.

4.1.1.2 Combined Sewer Systems

It may be impractical or un-economical to design a wastewater pumping station on a CSS to pump the expected 25-year peak wastewater flow, with the largest capacity pump out of operation. Under these conditions the following should be considered in determining the appropriate design capacity:

- The minimization of CSOs.
- The minimization of pumping station overflows as outlined in Section 4.3.

4.1.2 Accessibility

Wastewater pumping stations should be readily accessible by maintenance vehicles during all weather conditions. Multiple access points provide resilience against restricted or blocked access due to extreme events (e.g., flooding, forest fires, and blow down events) which may be worsened by climate change. Review of the potential impacts on the receiving environment and critical infrastructure will determine if a plant requires multiple access points. The plant should be located off street right-of-way in an appropriate area designated for pumping station purposes.

Consideration should also be given during design to accommodate future operation, maintenance, and safety concerns. This could include:

- Anchor points.
- Davit holders.
- Clearance for large equipment replacement.

4.1.3 Grit

The design of the wet wells should receive special attention and the discharge piping should be designed to prevent grit settling in pump discharge lines of pumps not operating. Pump components should be suitably designed for grit and pump speed should not exceed 1,800 RPM.

4.1.4 Sewer Entry

If more than one sewer enters the site of the pumping station, a junction manhole should be provided so that only a single sewer entry to the wet well is required.

4.1.5 Fencing

Pumping stations and associated facilities located in areas subject to vandalism or in areas warranting higher security may be fenced as a safety precaution. The fence should have an opening gate for entry of vehicles and equipment, and the gate should be kept locked to prevent vandalism.

4.1.6 Heating

Automatic heating may be required at pumping stations to prevent freezing in cold weather and to maintain a comfortable working temperature (there may be exceptions in the case of small below ground wet well or manhole type lift stations).

4.1.7 Piping System

The design of the pumping and piping systems should account for the potential of surge, water hammer, and special requirements for pump seals associated with wastewater service.

Suction and discharge piping should be sized to accommodate expected peak hourly flows with velocities ranging from 0.8 m/s to 2.0 m/s, where feasible velocities at the low end of the range are preferable.

Consideration should be given to providing access ports for such things as sampling, swabbing, and/or flushing discharge pressure gauge(s).

4.1.8 Electrical

All wiring should be in accordance with the requirements of the latest edition of *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations* and the local inspection authority.

Adequate heating should be installed to provide a minimum ambient temperature of 15°C to permit the provision of dehumidification equipment in the dry side of wet well/dry well pumping stations.

Electrical systems should be designed to meet the hazardous area classification of the space. This is of particular concern in spaces that are open or exposed to raw wastewater such as the wet well and building space above the wet well depending on the station design.

4.1.9 Lighting

Lighting levels should be provided in accordance with Illuminating Engineering Society (IES) recommended practice for similar area and use classifications.

4.1.10 Safety

The design and construction of all components of wastewater pumping stations should conform to the safety provisions of the occupational health and safety and construction safety legislation in the region where the pumping station is located.

4.1.11 Construction Materials

Due consideration should be given to the selection of materials and equipment because of the presence of hydrogen sulphide and other corrosive and inflammable gases, greases, oils, and other constituents present in wastewater.

4.2 Design

4.2.1 Types of Pumping Systems

The type of wastewater pumping station should be selected based on such considerations as:

- Reliability and serviceability.
- Operation and maintenance factors.
- Relationship to existing stations/equipment.
- Wastewater characteristics.

- Flow patterns and discharge.
- Long-term capital, operating, and maintenance costs.

For large main pumping stations, wet well/dry well type stations are recommended. For smaller stations and in cases for which wet well/dry well types are not feasible, wet well (submersible) pump stations may be used if pumps can be easily removed for replacement or repairs.

4.2.2 Structures

4.2.2.1 Separation

Wet and dry wells including their superstructure should be completely separated.

4.2.2.2 Equipment Removal

Provision should be made to facilitate removing pumps, motors, and other mechanical and electrical equipment.

4.2.2.3 Access

Suitable and safe means of access should be provided to dry wells of pump stations and to wet wells or to other parts of the building containing bar screens or mechanical equipment requiring inspection or maintenance. Stairways should be installed, with rest landings not to exceed 3.0 m vertical intervals.

4.2.3 Pumps & Pneumatic Injectors

4.2.3.1 Duplicate Units

At least two pumps or pneumatic ejectors should be provided. A minimum of three pumps should be provided for stations handling flows greater than 4,500 m^3/d.

4.2.3.2 Pump Protection

The need for and the type of screening facilities required for pumping stations varies with the characteristics of the wastewater, size of sewers, and the requirements of the operating authority. For wet well/dry well stations, it is an accepted practice to provide screening in the form of a manually or mechanically cleaned bar screen. Manually cleaned bar screens should be provided with minimum 38 mm clear openings in the inclined (60°) and horizontal bars. The maximum opening between bars should be less than the solids handling capacity of the smallest pump in the station. The vertical sides should be solid. The minimum width should be 600 mm. A drain platform should be provided for screenings. Proper attention should be given to channel design to maintain proper approach velocities for screens. The purpose of screening is to prevent the accumulation of solids in wet wells or pumping equipment that may disrupt operation or reliability of the pumping system.

Where areas have had their classification downgraded, ensure that suitable ventilation, controls, monitoring devices, alarms, building separations and other considerations are in accordance with applicable codes and requirements of local authorities.

Pumps handling separate sanitary wastewater from 750 mm, or larger diameter sewers should be protected by bar screens meeting the above requirements.

4.2.3.3 Pump Openings

Pumps should be capable of passing spheres of at least 75 mm in diameter. Pump suction and discharge openings should be at least 100 mm in diameter.

4.2.3.4 Priming

The pump should be so placed that under normal operating conditions it will operate under a positive suction head, except as specified in Section 4.2.10.

4.2.3.5 Electrical Equipment

The wet wells of wastewater pumping stations may occasionally contain flammable mixtures presenting a potentially hazardous (explosive) environment. As a minimum, electrical installations in these areas should comply with the requirements of the latest edition of *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations* or as otherwise required by the local inspection authority.

Provide area classifications clearly indicated on electrical drawings in accordance with the latest edition of *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations*. Drawings should indicate the full area that is classified.

4.2.3.6 Intake

Each pump should have an individual intake. Wet well design should be such as to avoid turbulence near the intake.

4.2.3.7 Constant Speed vs. Variable Speed Pump

In certain instances, such as pumping stations discharging directly into mechanical WWTPs or into other pumping stations, some means of flow pacing may be required. This can be provided by various means, depending upon the degree of flow pacing necessary. Where flow surges to treatment plants may be detrimental to the treatment process, variable speed control drives should be considered. If minor surges can be tolerated, two-speed pumps or multiple constant speed pumps can be used.

4.2.3.8 Controls

Control systems should be of the air bubbler type or the encapsulated float type. Where Programmable Logic Controllers (PLCs) form the basis of the station control system, consideration should be given to continuous level measurement via ultrasonic or submersible level transmitters. Pump control set-points are derived from the analog level signal in the PLC. For this type of installation, emergency start and stop float switches should be included to maintain station operation in the event of instrument failure.

Electrical systems and level control devices, located in the station wet wells, are to be suitably designed for the area classification. Float control should be positioned as per Section 4.2.5.5.

4.2.3.9 Alternation

Provisions should be made to automatically alternate the pumps in use. In the event of pump failure, the alternate pump should operate as the lead pump.

4.2.4 Valves

4.2.4.1 Suction Line

Suitable shut-off valves should be placed on the suction line of each pump except on submersible and vacuum-primed pumps.

4.2.4.2 Discharge Line

Suitable shut-off and check valves should be placed on the discharge line of each pump. The check valve should be located between the shut-off valve and the pump. Check valves should be suitable for the material being handled. Valves should be capable of withstanding the maximum anticipated surge pressure because of water hammer.

Where limited pump backspin will not damage the pump and low discharge head conditions exist, short individual force mains for each pump may be considered in lieu of discharge valves.

4.2.5 Wet Wells

4.2.5.1 Divided Wells

Where continuity of pumping station operation is important, consideration should be given to dividing the wet well into multiple sections, properly interconnected, to facilitate repairs and cleaning. Divided wet wells should also be considered for all pumping stations with capacities in excess of 100 L/s.

4.2.5.2 Pump Cycle

For any pumping station, the wet well should be of sufficient size to allow for a minimum of a 15-minute cycle time for each pump. For a two-pump station, the volume of the wet well in m^3, between pump start and pump stop should be 0.225 times the pumping rate of one pump, expressed in L/s. For other numbers of pumps, the required volume of the wet well depends upon the operating mode of the pumping units. Maximum recommended starts per hour are six for dry pit motors and 12 for submersible motors.

4.2.5.3 Size

Wet well size and control settings should be based on consideration of the volume required for:

- Pump cycling.
- The design fill time.
- Dimensional requirements to avoid turbulence problems.
- Vertical separation between pump control points.
- Inlet sewer elevation.
- Capacity required between alarm levels.
- Basement flooding.
- Overflow elevations, etc.

Wet wells should be designed to prevent septicity problems.

4.2.5.4 Floor Slope

The wet well floor should be sloped and employ design strategies to prevent solids deposition and grit accumulation. The horizontal area of the hopper bottom should be no greater than necessary for proper installation and function of the pipe inlet.

4.2.5.5 Float Controls

Float controls should be at least 300 mm vertically and 125 mm horizontally apart and positioned against a wall away from turbulent areas.

4.2.5.6 Pump Start Elevation

Low water level (pump shut-down) should be at least 300 mm or twice the pump suction diameter, whichever is greater, above the centre line of the pump volute. In cases where the selected stop elevation is below the top of the pump motor casing, ensure pump is supplied with adequate cooling to allow for continuous operation under partially submerged conditions.

4.2.5.7 Pump Stop Elevation

Low water level (pump shut-down) should be at least 300 mm or twice the pump suction diameter, whichever is greater, above the centre line of the pump volute.

4.2.5.8 Bottom Elevation

The bottom of the wet well should be no more than D/2, nor less than D/3 below the mouth of the flared intake where turned-down, bell-mouth inlets are used. In this case, “D” is the diameter of the mouth of the flared intake.

4.2.5.9 Air Displacement

Covered wet wells should have provisions for air displacement such as an inverted “j” tube or other means which vents to the outside.

4.2.5.10 Location of Valves

Valves should not be located in the wet well unless permitted by Regulatory Authority having jurisdiction. For dry well/wet well configuration, all valves should be installed in the dry well. For submersible pump stations it is recommended to have valves installed in a separate valve chamber. If valves must be located in the wet well, and it is permitted by the Regulatory Authority, provision should be made to allow for service of the valves and valves should be positioned above the emergency overflow level.

4.2.5.11 Wet Well Access Hatches

Hatches must be positioned directly above removable equipment. Hatches which are flush with the surrounding grade are to be equipped with a secondary protective grating device to provide fall-through protection. Access hatches should be provided with gas assist lift mechanism and should be lockable. Hatches are to open in a direction which allows access from the main access route.

4.2.6 Dry Wells

4.2.6.1 Dry Well Dewatering

A separate sump pump equipped with dual check valves should be provided in the dry wells to remove leakage or drainage, with the discharge above the overflow level of the wet well. A connection to the pump suction is also recommended as an auxiliary feature. Water ejectors connected to a potable water supply will not be approved. All floor and walkway surfaces should have an adequate slope to a point of drainage. Pump seal water should be piped to the sump.

4.2.6.2 Maintenance

The dry well should be equipped with a lifting beam to facilitate removal of pump motors. A roof hatch is recommended to provide access for removal of the entire pump and motor.

4.2.7 Ventilation

4.2.7.1 General

Adequate ventilation should be provided for all pump stations. Provide ventilation systems in accordance with the latest editions of *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations* and *NFPA 820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities*. Where the pump pit is below the ground surface, mechanical ventilation is required, so arranged as to independently ventilate the dry well and the wet well. There should be no interconnection between the wet well and dry well ventilation systems. Ventilation must avoid dispensing contaminants throughout other parts of the pumping station, and vents should not open into a building or connect with a building ventilation system.

4.2.7.2 Air Inlets & Outlets

In dry wells over 4.6 m deep, multiple inlets and outlets are desirable. Dampers should not be used on exhaust or fresh air ducts and fine screens or other obstructions in air ducts should be avoided to prevent clogging.

4.2.7.3 Electrical Controls

Switches for operation of ventilation equipment should be marked and located conveniently. All intermittently operated ventilation equipment should be interconnected with the respective pit lighting system. Consideration should also be given to automatic controls where intermittent operation is used. The manual lighting ventilation switch should override the automatic controls.

Provide flow monitoring and controls as required in accordance with the latest edition of *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations* and requirements from local authorities.

4.2.7.4 Fans, Heating, & Dehumidification

The fan wheels should be fabricated from non-sparking material. Automatic heating and dehumidification equipment should be provided in all dry wells. The electrical equipment and components should meet the requirements in Section 4.2.3.5.

4.2.7.5 Wet Wells

Ventilation may be either continuous or intermittent and should be controlled in line with the latest edition of *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations* and requirements from local authorities. Fresh air should be forced into the wet well, by mechanical means, at a point, typically, 300 mm above the expected high liquid level. There should be a provision for automatic blow-by to elsewhere in the well, should the fresh air inlet become submerged.

4.2.7.6 Dry Wells

Ventilation may be either continuous or intermittent and should be controlled in line with the latest edition of *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations* and *NFPA 820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities*. Ventilation should be forced into the dry well at a point, typically, 150 mm above the pump floor, and allowed to escape through vents in the roof superstructure.

4.2.8 Flow Measurement

Suitable devices for measuring wastewater flow should be provided at all pumping stations. Indicating, totalizing, and recording flow measurement should be provided at pumping stations with a 50 L/s or greater design peak hourly flow. Elapsed time meters used in conjunction with pumping rate tests may be acceptable for pumping stations with a design peak hourly flow up to 50 L/s.

4.2.9 Water Supply

There should be no physical connection between any potable water supply and a wastewater pumping station which under any conditions might cause contamination of the potable water supply. If a potable water supply is brought to the station, it should be protected with a suitable backflow prevention device (see Section 5.7.2.5).

4.2.10 Suction Lift Pumps

4.2.10.1 General

Suction lift pumps should be of the self-priming or vacuum-priming type and should meet the applicable requirements of Section 4.2. Suction lift pump stations using dynamic suction lifts exceeding the limits outlined in the following sections may be approved by the appropriate Regulatory Authority upon submission of factory certification of pump performance and detailed calculations indicating satisfactory performance under the proposed operating conditions. Such detailed calculations must include static suction lift as measured from “lead pump off” elevation to centre line of pump suction, friction and other hydraulic losses of the suction piping, vapour pressure of the liquid, altitude correction, required net positive suction head and a safety factor of at least 1.8 m.

The pump equipment compartment should be above grade or offset and should be effectively isolated from the wet well to prevent the humid and corrosive sewer atmosphere from entering the equipment compartment. Wet well access should not be through the equipment compartment. Valves should not be located in the wet well.

4.2.10.2 Self-Priming Pumps

Self-priming pumps should be capable of rapid priming and re-priming at the “lead pump on” elevation. Such self-priming and re-priming should be accomplished automatically under design operating conditions. Suction piping should not exceed the size of the pump suction and should not exceed 7.6 m in total length. Priming lift at the “lead pump on” elevation should include a safety factor of at least 1.2 m from the maximum allowable priming lift for the specific equipment at design operating conditions. The combined total of dynamic suction lift at the “pump off” elevation and required net positive suction head at design operating conditions should not exceed 6.7 m.

4.2.10.3 Vacuum-Priming Pumps

Vacuum-priming pump stations should be equipped with dual vacuum pumps capable of automatically and completely removing air from the suction lift pump. The vacuum pumps should be adequately protected from damage due to wastewater. The combined total of dynamic suction lift at the “pump off” elevation and required net positive suction head at design operating conditions should not exceed 6.7 m.

4.2.11 Submersible Pump Stations

4.2.11.1 General

A submersible pump station in this document is defined as having one chamber for the collection of wastewater which also contains the pumps.

Submersible pump stations should meet the applicable requirements under Section 4.2 except as modified in this section.

4.2.11.2 Construction

Submersible pumps and motors should be designed specifically for raw wastewater use, including totally submerged operation during a portion of each pumping cycle. An effective method to detect shaft seal failure or potential seal failure should be provided and the motor should be of squirrel-cage type design without brushes or other arc-producing mechanisms.

4.2.11.3 Pump Removal

Submersible pumps should be readily removable and replaceable without dewatering the wet well or disconnecting any piping in the wet well.

4.2.11.4 Wet Wells

See Section 4.2.5 for the layout of wet wells.

4.2.11.5 Mixing for Wet Wells

Consideration should be given to mixing of the wet well using flushing mechanisms which are attached to the submersible pumps and readily accessible for maintenance and inspection.

4.2.11.6 Power Supply

Pump power cables, control and alarm circuits should be designed to provide strain relief and to allow disconnection from outside the wet well. Cable terminations should be made outside the wet well in enclosures suitably rated for the ambient environment. Electrical equipment should be positioned above flood elevations, including climate change considerations. Electrical equipment should be installed in a manner that protects it from extreme weather, which may be worsened by climate change.

Power supply equipment located outside/above ground are vulnerable to extreme weather events. The impacts of climate change may result in an increased risk of power outages resulting from potential increases in the frequency and intensity of weather events (e.g., hurricanes and other large wind events). If feasible, the System

Owner should locate electrical service equipment underground to provide resiliency from extreme weather events and the impacts of climate change.

4.2.11.7 Controls

The pump controller should be located outside the wet well. Conduit sealing is required at the entry to field junction boxes or pump controllers and should be in accordance with the specific requirements of the Inspection Authority. If conventional conduit EY type seal fittings are utilized, they should be located such that the pump power and/or control cables can be removed and electrically disconnected without disturbing the seal.

4.2.11.8 Power Cables

Pump motor cables should be designed for flexibility and serviceability under conditions of extra hard usage and should meet the requirements of the latest edition of *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations*. The ground fault system should be used to de-energize the circuit in the event of any failure in the electrical integrity of the cable.

4.2.11.9 Valves

Required valves should be located in a separate valve pit unless their placement within the submersible pump station itself is acceptable to the jurisdiction having authority. Accumulated water should be drained to the wet well or to the soil. If the valve pit is drained to the wet well, an effective method should be provided to prevent wastewater from entering the pit during surcharged wet well conditions.

4.2.12 Cathodic Protection

Steel fabricated pumping stations should require cathodic protection for corrosion control. Impressed current or magnesium anode packs are generally used for this purpose in conjunction with a suitable protective coating on underground surfaces, applied in accordance with the manufacturer’s directions. The unit should be electrically isolated by dielectric fittings placed on inlet and outlet pipes, anchor bolts and electrical conduit boxes.

Upon completion of the installation, the capability of the anti-corrosion system should be verified by instrumentation. Such inspection should be carried out by a person approved by the reviewing agencies.

4.2.13 Alarm Systems

Alarm systems should be provided for pumping stations. The alarm should be activated in cases of:

- Power failure.
- Pump failure.
- Use of the lag pump.
- Unauthorized entry.
- Emergency high level reached.
- Overflow event.
- Low level reached.
- Or any cause of pump station malfunction.

Pumping station alarms should be telemetered, including identification of the alarm condition, to a municipal facility that is monitored 24 hours a day. If such a facility is not available and 24-hour holding capacity is not provided, the alarm may be telemetered to municipal offices during normal working hours or to the home of the person(s) in charge of the pumping station during off-duty hours. Audio visual alarm systems with a self-contained power supply may be acceptable in some cases in lieu of the telemetering system outlined above depending upon location, station holding capacity, and inspection frequency.

4.3 Emergency Operation

The objective of the emergency operation is to prevent the discharge of raw or partially treated wastewater to any waters and to protect public health by preventing back-up of wastewater and subsequent discharge to basements, streets, and other public and private property.

4.3.1 Overflow Prevention Methods

A satisfactory method should be provided to prevent or minimize overflows. The following methods should be evaluated on a case-by-case basis:

- Provision of standby generator with automatic switchover, sized to operate the station at maximum capacity. Where station sizing allows, and with the approval of Regulatory Authorities, an outlet for connection of a portable generator may be provided. The risk of power outages may increase due to potential increases in extreme weather events (e.g., hurricanes and other large wind events) resulting from climate change.
- Storage capacity, including trunk sewers, for retention of wet weather flows (storage basins must be designed to drain back into the wet well or collection system after the flow recedes).
- An in-place or portable pump, driven by an internal combustion engine meeting the requirements of Section 4.3.3 below, capable of pumping from the wet well to the discharge side of the station.

4.3.2 Overflow

In some jurisdictions, provisions should be made for disinfection of the overflow raw wastewater. System Owners should consult with Regulatory Authority to determine provincial requirements. Where possible, it is recommended to include means to retain floatable material in the system which can be accomplished using baffles. The overflow facilities should be alarmed and equipped to indicate frequency and duration of overflows and designed to permit manual flow measurement. Where the operator is signatory to a *Shellfish Conditional Area Management Plan (Management of shellfish harvesting in areas next to wastewater treatment plants* from Fisheries and Oceans Canada), notification and reporting requirements of the plan should be met. All overflows should be recorded and reported to the Regulatory Authorities.

4.3.3 Engine-Driven Equipment Requirements

The following general requirements should apply to all internal combustion engines used to drive auxiliary pumps, service pumps through special drives, or electrical generating equipment.

4.3.3.1 Engine Protection

The engine must be protected from operating conditions that would result in damage to equipment. Unless continuous manual supervision is planned, protective equipment should be capable of shutting down the engine and activating an alarm on site and as provided in Section 4.2.13. Protective equipment should monitor for conditions of low oil pressure and overheating, except that oil pressure monitoring will not be required for engines with splash lubrication.

4.3.3.2 Size

The engine should have adequate rated power to start and continuously operate all connected loads.

4.3.3.3 Fuel Type

Reliability and ease of starting, especially during cold weather conditions, should be considered in the selection of the type of fuel.

4.3.3.4 Engine Ventilation

The engine should be located above grade with adequate ventilation of fuel vapours and exhaust gases.

4.3.3.5 Routine Start-Up

All emergency equipment should be provided with instructions indicating the need for regular starting and running of such units at full loads.

4.3.3.6 Protection of Equipment

Emergency equipment should be protected from damage at the restoration of regular electrical power.

4.3.4 Engine-Driven Pumping Equipment

Where permanently installed or portable engine-driven pumps are used, the following requirements in addition to general requirements should apply.

4.3.4.1 Pumping Capacity

Engine-driven pumps should meet the design pumping requirements unless storage capacity is available for flows in excess of pump capacity. Pumps should be designed for anticipated operating conditions, including suction lift, if applicable.

4.3.4.2 Operation

The engine and pump should be equipped to provide automatic start-up and operation of pumping equipment. Provisions should also be made for manual start-up. Where manual start-up and operation is justified, storage capacity and alarm systems must meet the requirements of Section 4.3.4.3.

4.3.4.3 Portable Pumping Equipment

Where part or all of the engine-driven pumping equipment is portable, sufficient storage capacity to allow time for detection of pump station failure and transportation and hook up of the portable equipment should be provided. A riser from the force main with quick-connect coupling and appropriate valving should be provided to hook up portable pumps.

4.3.5 Engine-Driven Generating Equipment

Where permanently installed or portable engine-driven generating equipment is used, the following requirements in addition to general requirements should apply.

4.3.5.1 Generating Capacity
Generating unit size should be adequate to provide power for pump motor starting current and for lighting, ventilation, and other auxiliary equipment necessary for safe and proper operation of the pumping station. Fuel storage requirement for emergency generator should be sized for the anticipated outage in the future, recognizing the potential for increase in the length of outage and potential loss of access for fuel delivery. The risk of power outages may increase due to potential increases in extreme weather events (e.g., hurricanes and other large wind events) resulting from climate change. The System Owner should ensure that the fuel supplier has back-up power during a power outage and can deliver fuel during the outage.

The operation of only one pump during periods of auxiliary power supply must be justified. Such justification may be made based on maximum anticipated flows relative to single-pump capacity, anticipated length of power outage, and storage capacity. Special sequencing controls should be provided to start pump motors unless the generating equipment has capacity to start all pumps simultaneously with auxiliary equipment operating.

4.3.5.2 Operation
Provisions should be made for automatic and manual start-up and load transfer. The generator must be protected from operating conditions that would result in damage to equipment. Provisions should be considered to allow the engine to start and stabilize at operating speed before assuming the load. Where manual start-up and transfer is justified, storage capacity and alarm systems must meet requirements of Section 4.3.4.3.

4.3.5.3 Portable Generating Equipment
Where portable generating equipment or manual transfer is provided, sufficient storage capacity to allow time for detection of pump station failure, transportation, and connection of generating equipment should be provided. Transportation of portable equipment to the site may be impaired due to potential increases in the occurrence of extreme weather events (e.g., flooding and large storm events) resulting from climate change, which should be considered when sizing storage capacity. The use of special electrical connections and double throw switches are recommended for connecting portable generating equipment.

4.4 Instructions & Equipment
The operating authority of wastewater pumping stations should be supplied with a complete set of operational instructions, including emergency procedures, maintenance schedules, tools and such spare parts as may be necessary.

4.5 Force Mains
4.5.1 Velocity
At design average flow, a cleansing velocity of at least 0.6 m/s should be maintained.

4.5.2 Air Relief Valve & Blowoff
An automatic air relief valve should be placed at high points in the force main to prevent air locking. Drain or blow-off valves should be provided at all low points in pressure sewers. Additional air and vacuum relief valves may be required at other locations along the force main such as inflection points where there is a change in pipe slope.

4.5.3 Termination

Force mains should enter the gravity sewer system at a point not more than 0.6 m above the flow line of the receiving manhole. A 45° bend may be considered to direct the flow downward.

4.5.4 Design Pressure

The force main and fittings, including reaction blocking, should be designed to withstand normal pressure and pressure surges.

4.5.5 Size

Force mains should be sized to provide sufficient flow velocity required capacity at the available head and to withstand operating pressures as outlined in Sections 4.5.1 and 4.5.4. In general, force mains should be a minimum of 100 mm in diameter.

4.5.6 Slope & Depth

Force main slope does not significantly affect the hydraulic design or capacity of the pipeline itself. Under no circumstance, however, should any force main be installed at zero slope. Zero slope installation makes line filling and pressure testing difficult and promotes accumulation of air and wastewater gases.

A force main should have a minimum cover of 1.8 m.

4.5.7 Special Construction

Force main construction near watercourses or used for aerial crossing should meet applicable requirements of Sections 3.12 to 3.14.

4.5.8 Design Friction Losses

Friction losses through force mains should be based on the Hazen Williams formula or another acceptable method. When the Hazen Williams formula is used, the following values for "C" should be used for design.

Unlined iron or steel	-	100
All other	-	120

When initially installed, force mains will have a significantly higher "C" factor. The "C" factor of 120 should be considered in calculating maximum power requirements for smooth pipe.

4.5.9 Separation from Water Mains

Water mains and wastewater force mains are to be installed in separate trenches. The soil between the trenches should be undisturbed. Force mains crossing water mains should be laid to provide a minimum vertical distance of 450 mm between the outside of the force main and the outside of the water main. The water main should be above the force main. At crossings, one full length of water pipe should be located so both joints will be as far from the force main as possible. Special structural support for the water main and the force main may be required.

4.5.10 Identification

Where force mains are constructed of material which might cause the force main to be confused with potable water mains, the force main should be appropriately identified.

4.6 Testing

4.6.1 General

The entire length of a force main should be tested for leakage. If the length of a force main exceeds 400 m, the allowable leakage must not exceed the allowable leakage for a similar force main 400 m in length. All valves in the force main must be opened immediately prior to testing.

4.6.2 Leakage Test

The force main should be filled with water, and a test pressure of 1,035 kPa (150 psi) or equal to 1.5 times the working pressure should be applied, measured at the lowest point in the test section. The pressure should be maintained by pumping water from a suitable container of known volume. The amount of water used for a period of 2 hours should be recorded.

4.6.3 Allowable Leakage

Allowable leakage for a force main should be determined by the following formula:

$$L = \frac{(SD) \times P^{0.5}}{727{,}500}$$

Where:

L	=	Allowable leakage in L/hour
S	=	Length of pipe in m
D	=	Nominal diameter of pipe in mm
P	=	Test pressure in kPa

Allowable leakage for closed metal seated valves is 1.2 mL per mm of nominal valve diameter per hour. The maximum test section should be 400 m or as directed by the Regulatory Authority having jurisdiction.

4.7 References

American Water Works Association. *M28 Rehabilitation of Water Mains*. AWWA, Denver, 2001.

Canadian Centre for Occupational Health and Safety, Emergency Showers and Eyewash Stations, January 2017. <http://www.ccohs.ca/oshanswers/safety_haz/emer_showers.html>.

Canadian Standards Association. *Canadian Electrical Code C22.1:21*. CSA Group, 2021.

National Fire Protection Association. 20 *Standard for the Installation of Stationary Pumps for Fire Protection*. NFPA, 2022

National Fire Protection Association. *820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities*. NFPA, 2020

Ontario Ministry of Natural Resources, *Manual of Instruction - Aquatic Habitat Inventory Surveys*, 1987. <https://docs.ontario.ca/documents/2658/stdprod-103352.pdf>.NFPA. 820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities. NFPA, 2020

Ontario Ministry of the Environment, *Design Guidelines for Sewage Works*, May 2019. <https://www.ontario.ca/document/design-guidelines-sewage-works-0>.

Standards Council of Canada. *CSA W204:19 Flood Resilient Design of New Residential Communities*. Toronto: CSA Group, December 2019. <https://www.csagroup.org/store/product/CSA%20W204%3A19/>.

Tchobanoglous, George, Burton, Franklin L., Stensel, H. David, and Metcalf & Eddy. *Wastewater Engineering: Treatment and Reuse*. 4th Ed. / Revised by George Tchobanoglous, Franklin L. Burton, H. David Stensel. ed. McGraw-Hill Series in Civil and Environmental Engineering. Boston; Montreal: McGraw-Hill, 2003.

Vesilind P. *Wastewater Treatment Plant Design*. Water Environment Federation (WEF), 2003.

Table of Contents

Chapter 5 Wastewater Treatment Plants

The latest edition of *CSA S900.1.18 Climate change adaptation for wastewater treatment plants* should be considered when designing a WWTP.

5.1 Definition of Wastewater Treatment Plant

Wastewater treatment plant means a facility for the treatment of sanitary wastewater with a discharge of the treated effluent off the site, or effluent dispersal (subsurface or surface irrigation).

5.2 Effluent Quality

The required degree of wastewater treatment should be based on the effluent requirements and water quality standards established by the Regulatory Authority and/or appropriate federal regulations, including discharge permit requirements. Treatment should be provided in connection with all sewer installations. The engineer should confer with the Regulatory Authority to establish provincial requirements before proceeding with the design of wastewater infrastructure. All effluent discharges, regardless of daily volumes, require a permit prior to release. For all systems with average daily volumes of 100 m^3/day or greater, System Owners should refer to WSER requirements for minimum effluent quality limits (certain exceptions apply).

The typical level of treatment required for any new treatment plant in Atlantic Canada is secondary treatment with disinfection. Higher levels of treatment may be required based on the risk to the receiving water, which the Regulatory Authority may require to be evaluated by carrying out an Environmental Risk Assessment (ERA). For the procedure for carrying out these assessments, refer to the latest editions of *Guidance on the Site-Specific Application of Water Quality Guidelines in Canada: Procedures for Deriving Numerical Water Quality Objectives* from the Canadian Council of Ministers of the Environment, *Canada-wide Strategy for the Management of Municipal Wastewater Effluent Technical Supplement 2: Environmental Risk Management Framework and Guidance* from the Canadian Council of Ministers of the Environment, and *Canada-wide Strategy for the Management of Municipal Wastewater Effluent Technical Supplement 3: Standard Method and Contracting Provisions for the Environmental Risk Assessment* from the Canadian Council of Ministers of the Environment.

5.3 Site Considerations

5.3.1 Climate Change Risk Assessment

A risk assessment should be completed for the selected site which considers potential climate change impacts to the WWTP. Refer to Chapter 2 of these Guidelines and the latest edition of *CSA S900.1.18 Climate change adaptation for wastewater treatment plants* for further guidance on conducting a risk assessment.

5.3.2 Plant Location

The following items should be considered when selecting a plant site:

- Proximity to residential areas.
- Direction of prevailing winds.

- Accessibility, including climate change considerations (e.g., disruptions to roadways from flooding, blow down events, and forest fires).
- Area available for expansion.
- Local zoning requirements.
- Local soil characteristics, geology, hydrology, and topography available to minimize pumping.
- Access to receiving water.
- Downstream uses of the receiving water (including but not limited to shellfish harvesting areas, public swimming areas, and drinking water supply intakes).
- Compatibility of treatment process with the present and planned future land use (including noise, potential odours, air quality, and anticipated sludge processing and disposal techniques).
- Proximity to surface water supplies and water wells.
- Proximity to environmentally sensitive areas such as protected wetlands.
- Storm surge (including climate change considerations).
- Flood protection (including climate change considerations).
- Other climate change impacts (e.g., impact of sea level rise on plant hydraulics).

Some of these items have recommended guidelines discussed below. Appropriate measures should be taken to minimize adverse impacts where a site does not meet recommended guidelines for these items. Refer to Chapter 2 of these Guidelines and the latest edition of *CSA S900.1.18 Climate change adaptation for wastewater treatment plants* for further information regarding adaptation strategies.

5.3.2.1 Separation Distances

Separation distances should be designed to prevent the occurrence of objectionable odours in residential areas and surface water and groundwater contamination, when WWTPs are operated normally and within designed capacities. They should not be designed to accommodate unusual upset conditions that may occur from time to time. Lesser separation distances may be approved upon receipt of permission from adjacent or nearby property owners.

Separation distances will be measured from the proposed odour producing source or component to the nearest neighbouring lot line. Specific separation distances are as follows:

- Mechanical plants (including aerated stabilization ponds) should be located at least:
  - 150 m from residences.
  - 30 m from commercial-industrial developments.
  - 30 m from the nearest property lines.
  - Under special circumstances a lesser separation distance to residences may be adopted, provided that odour control equipment is provided at the plant.
- Waste stabilization ponds (not mechanically aerated) should be located at least 150 m from residences or as determined by the Regulatory Authority having jurisdiction (some jurisdictions may require 300 m setback).
- Recirculating sand or textile filters should be located at least:
  - 30 m from potable water supply wells.
  - 100 m from water supply wells immediately down slope.
  - 3.0 m from any lot boundary.
  - 9.0 m up slope of any lot boundary.
- For infiltration and irrigation separation distances see Sections 11.2.3.6 and 11.3.2.2, respectively.

5.3.3 Flood Protection

The treatment works structures, electrical and mechanical equipment should be protected from physical damage from the 1-in-100-year flood extent. Treatment works should remain fully operational and accessible during the 1-in-25-year flood. Designers should account for climate change when determining flood levels (e.g., potential increases in the frequency and intensity of extreme precipitation events, sea level rise and storm surge). Refer to Chapter 2 of these Guidelines and the latest edition of *CSA S900.1.18 Climate change adaptation for wastewater treatment plants* for further guidance. This applies to new construction and to existing facilities undergoing major modifications. Flood plain regulations of provincial and federal agencies should be considered.

5.3.4 General Plant Layout

The general arrangement of the plant within the site should consider the subsurface conditions and natural grades to provide the necessary facilities at minimum cost, as well as climate change considerations. Refer to Chapter 2 of these Guidelines and the latest edition of *CSA S900.1.18 Climate change adaptation for wastewater treatment plants* for further guidance.

In the layout of the plant, the designer should orient the buildings to provide adequate allowances for future linear expansions of the various treatment sections and orient the plant so that the best advantage can be taken of the prevailing wind and weather conditions to minimize odour, misting and freezing problems, and energy consumption. The plant layout should also allow for the probability of snow drifting with entrances, roadways, and open tankage located so that the effect of snow drifting on operations will be minimized.

It is not recommended that construction of any of the facilities be near a shoreline, except where this is unavoidable. Suitable measures must be taken to adequately protect the structures from the impacts of wave action, shoreline erosion, sea level rise, and storm surge.

Within the constraints mentioned above, the designer should work towards a plant layout where the various processing units are arranged in a logical progression to avoid the necessity for major pipelines or conduits to transmit wastewater, sludge, or chemicals from one module to the next, and also to arrange the plant layout to provide for convenience of operation and ease of flow splitting for proposed and future treatment units. Wherever possible, the plant layout should allow for gravity flow between unit processes to reduce the need for additional pumping operations.

Key risks to site accessibility and parking which may restrict site access and operation should be considered in the design, including climate change considerations (e.g., flooding, blow down events, and forest fires). Where site roadways are provided for truck access, the road design should be sufficient to withstand the largest anticipated delivery or disposal vehicles with due allowance for vehicle turning and forward exit from the site.

To avoid the dangers of high voltage lines crossing the site, a high voltage pole should be located at the property line. If the distance from the terminal pole to the control building is short, the step-down transformer should be located at the terminal pole. If the distance from the terminal pole to the control building is long, the transformer should be located adjacent to the building, and the high voltage connections should be brought by underground cable to the pothead or cable termination at the transformer.

Wastewater treatment works sites should be adequately fenced, signed, and posted to prevent unauthorized access.

5.3.5 Provision for Future Expansion

In addition to the general site considerations outlined in Section 5.3.4, there are a number of allowances needed to provide for economical and practical expansion of the WWTPs. Key provisions include:

- Design of on-site pumping stations such that their capacity can be increased and/or parallel facilities constructed without the need for major disruption of the plant's operation.
- Layout and sizing of channels and plant piping such that additional treatment units can be added or increases in loading rates accommodated. Similarly, the layout of buildings and tankage should accommodate the location of the future stages of expansion.
- Space provision within buildings to provide for replacement of equipment with larger capacity units. This is particularly important with equipment such as pumps, blowers, boilers, heat exchangers, etc. Adequate working space should be provided around equipment, and provision made for the removal of equipment.
- Sizing of inlet and outlet sewers to account for the ultimate plant capacity. Provided that problems will not occur with excessive sedimentation in the sewers, these sewers should be sized for the ultimate condition. With diffused outfalls, satisfactory port velocities can often be obtained by blocking off ports which will not be required until subsequent expansion stages.

5.4 General Design Requirements

5.4.1 Type of Treatment

A process should be capable of providing the necessary treatment and effluent discharge control to protect the adjacent and receiving environment.

As a minimum, the following items should be considered in the selection of the type of treatment:

- Present and future effluent requirements.
- Location of and local topography of the plant site.
- Space available for future plant construction.
- The effects of industrial wastes likely to be encountered.
- Ultimate disposal of sludge.
- System capital costs.
- System operating and maintenance costs, including basic energy requirements.
- Process complexity governing operating personnel requirements.
- Environmental impact on present and future adjacent land use.
- Wastewater characteristics and the results of any treatability or pilot plant studies.
- Reliability of the process and the potential for malfunctions or bypassing needs.
- Climate change considerations such as the impacts of sea level rise on the hydraulics of the treatment plant and its outfall.

5.4.2 Engineering Data for New Process Evaluation

The policy of the Regulatory Authority is to encourage rather than obstruct the development of any methods or equipment for the treatment of wastewater. The lack of inclusion in these standards of some types of wastewater treatment processes or equipment should not be construed as precluding their use. The Regulatory Authority may approve other types of wastewater treatment processes and equipment under the condition that the operational reliability and effectiveness of the process or device should have been demonstrated with a suitably sized prototype unit operating at its design load conditions, to the extent required.

The Regulatory Authority may require the following:

- Monitoring observations, including tests results and engineering evaluations, demonstrating the efficiency of such processes.
- Detailed description of the test methods.
- Testing, including appropriately composite samples, under various ranges of strength and flowrates (including diurnal variations) and waste temperatures over a sufficient length of time to demonstrate performance under climatic and other conditions which may be encountered in the area of the proposed installations.
- Other appropriate information.

The Regulatory Authority may require that appropriate testing be conducted, and evaluations be made under the supervision of a competent process engineer other than those employed by the manufacturer, patent holder, or developer.

5.4.3 Design Period

The design period should be clearly identified in the Design Report as required in Chapter 1.

Factors which will have an influence on the design period of wastewater treatment works include the following:

- Population growth rates.
- Prevailing financing interest rates.
- Inflation rates.
- Ease of expansion of facilities.
- Time requirements for design and construction or expansion.

Wherever possible, WWTPs should be designed for the flows expected to be received 20 years hence, under normal growth conditions. In certain cases, where it can be shown that staging of construction will be economically advantageous, lesser design periods may be used provided it can be demonstrated that the required capacity can be "online" when needed. The impacts of climate change at full build out should be considered.

5.4.3.1 Treatment Plant Design Capacity

The WWTP design capacity is the design average flow at the design average BOD_5.

The plant design flow selected should meet the appropriate effluent and water quality standards that are set forth in the discharge permit from the province, as well as the requirements of the federal WSER legislation, for all systems with average daily volumes of 100 m^3/day or greater (WSER minimum, certain exceptions apply). The design of treatment units that are not subject to peak hourly flow requirements should be based on the design average flow. For plants subject to high wet weather flows or overflow detention pumpback flows, the design maximum day flows that the plant is to treat on a sustained basis should be specified. The impacts of climate change on sea level, storm surge, and freshwater flood elevations are to be considered in the hydraulic design of the treatment plant.

5.4.3.2 Hydraulic Capacity

5.4.3.2.1 Hydraulic Flow Definitions & Identification

The following flows for the design year should be identified and used as a basis for design for sewers, lift stations, WWTPs, treatment units, and other wastewater handling facilities. Where any of the terms defined in this section are used in these design standards, the definition contained in this section applies.

The **Design Average Flow** is the average of the daily volumes to be received for the continuous 12-month period expressed as a volume per unit time, however, the design average flow for facilities having critical seasonal high hydraulic loading periods (e.g., recreational areas, cottages, campuses, and industrial/commercial facilities) should be based on the daily average flow during the seasonal period.

The **Design Maximum Day Flow** is the largest volume of flow to be received during a continuous 24-hour period expressed as a volume per unit time.

The **Design Peak Hourly Flow** is the largest volume of flow to be received during a 1-hour period expressed as a volume per unit time.

The **Design Peak Instantaneous Flow** is the instantaneous maximum flowrate to be received.

The **Design Minimum Day Flow** is the smallest volume of flow to be received during a 24-hour period during dry weather when infiltration/inflow are at a minimum, expressed as a volume per unit time.

Initial low flow conditions must be evaluated in the design to minimize operational problems with freezing, septicity, flow measurements and solids settling. The design peak hourly flows must be considered in evaluating unit processes, pumping, piping, etc.

- The sizing of WWTPs receiving flows from existing wastewater collection systems should be based on projections made from measured flow data.
- At least 1 year’s flow data should be taken as the basis for determining the various critical flow conditions.
- The probable degree of accuracy of data and projections should be evaluated. This reliability estimation should include an evaluation of the accuracy of existing data, as well as an evaluation of the reliability of estimates of flow reduction anticipated due to I&I reduction or flow increases due to elimination of sewer bypasses and backups.
- Critical data and methodology used should be included. It is recommended that graphical displays of critical peak wet weather flow data, be included for a sustained wet weather flow period of significance to the project.

5.4.3.2.2 Hydraulic Capacity for Wastewater Facilities to Serve New Collection Systems

- The sizing of WWTPs receiving flows from new wastewater collection systems should be based on an Average Daily Flow (ADF) of 380 L/cap, plus wastewater flow from industrial plants and major institutional and commercial facilities unless water use data or other justification upon which to better estimate flow is provided.

- The 380 L/cap·d figure should be used in conjunction with an extraneous flow allowance (see Section 3.3) intended to cover infiltration.
- If the new collection system is to serve existing development the likelihood of I&I contributions from existing service lines and non-wastewater connections to those service lines should be evaluated and wastewater facilities designed accordingly.

5.4.3.2.3 High Wet Weather Flows

If unusually high flows are encountered during wet weather periods, a thorough investigation of the collection system should be made and a program for corrective action initiated.

5.4.3.2.4 Flow Equalization

Facilities for the equalization of flows and organic shock loads should be considered at all plants which are critically affected by surge loadings. The sizing of the flow equalization facilities should be based on data obtained herein and from Section 3.3.

5.4.3.3 Organic Design Loads

5.4.3.3.1 Organic Load Definitions & Identification

The following organic loads for the design year should be identified and used as a basis for design of WWTPs. Where any of the terms defined in this section are used in these design standards, the definition contained in this section applies.

Biochemical Oxygen Demand

The BOD_5 is defined as the amount of oxygen required to stabilize biodegradable organic matter under aerobic conditions within a 5-day period in accordance with *Standard Methods for the Examination of Water and Wastewater* from the American Public Health Association, the American Water Works Association, and the Water Environment Federation. Total 5-day Biochemical Oxygen Demand ($TBOD_5$) is equivalent to BOD_5 and is sometimes used in order to differentiate carbonaceous plus nitrogenous oxygen demand from strictly carbonaceous oxygen demand.

The Carbonaceous 5-day Biochemical Oxygen Demand ($CBOD_5$) is defined as BOD_5 less the nitrogenous oxygen demand of the wastewater. See *Standard Methods for the Examination of Water and Wastewater* from the American Public Health Association, the American Water Works Association, and the Water Environment Federation.

Design Average BOD_5

The design average BOD_5 is generally the average of the organic load to be received for a continuous 12-month period for the design year expressed as weight per day, however, the design average BOD_5 for facilities having critical seasonal high loading periods (e.g., recreational areas, cottages, campuses, and industrial/commercial facilities) should be based on the average organic load to be received during the seasonal period.

Design Maximum Day BOD_5

The design maximum day BOD_5 is the largest amount of organic load to be received during a continuous 24-hour period expressed as weight per day.

Design Peak Hourly BOD_5
The design peak hourly BOD_5 is the largest amount of organic load to be received during a 1-hour period expressed as weight per day.

5.4.3.3.2 Design of Organic Capacity of Wastewater Treatment Plants to Serve Existing Collection Systems

- Projections should be made from actual waste load data to the extent possible. When sampling, consideration should be given to flow patterns for institutions, schools, motels, etc.
- Projections should be compared to Section 5.4.3.3.3 and account for significant variations from those values.
- Impact of industrial sources should be documented. For projects with significant industrial contributions, evidence of adequate pre-treatment strategies should be included along with documentation that industries are aware of the pre-treatment limitations and user costs associated with the project. Documentation of the individual industrial participation in the project plan including user charges should be provided.
- Septage and leachate may contribute significant organic load and other materials which can cause operational problems and non-compliance with permit limitations. If septage or leachate is to be discharged to the WWTP, consult the Regulatory Authority.

5.4.3.3.3 Design of Organic Capacity of Wastewater Treatment Plants to Serve New Collection Systems

- Projections should be made from actual waste load data to the extent possible. When sampling, consideration should be given to flow patterns for institutions, schools, motels, etc.
- Projections should be compared to Section 5.4.3.3.3 and account for significant variations from those values.
- Impact of industrial sources should be documented. For projects with significant industrial contributions, evidence of adequate pre-treatment strategies should be included along with documentation that industries are aware of the pre-treatment limitations and user costs associated with the project. Documentation of the individual industrial participation in the project plan including user charges should be provided.
- Septage and leachate may contribute significant organic load and other materials which can cause operational problems and non-compliance with permit limitations. If septage or leachate is to be discharged to the WWTP, consult the Regulatory Authority.

5.4.3.4 Shock Effects

The shock effects of high concentrations and diurnal peaks for short periods of time on the treatment process, particularly for small treatment plants, should be considered.

5.4.3.5 Design Capacity of Various Plant Components (Without Flow Equalization)

In general, all components of mechanical WWTPs should be hydraulically capable of handling the anticipated peak wastewater flowrates without overtopping channels and/or tankage. From a process point-of-view, however, the design of various sections of WWTPs should be based upon the following hydraulic, organic, and inorganic loading rates:

Wastewater Pumping Stations

- Peak hourly flow.

Screening

- Peak hourly flow.

Grit Removal

- Peak hourly flow.

Primary Sedimentation

- Peak hourly flow.
- Flowrate.
- Peak solids loading rate.

Aeration (without nitrification)

- Average BOD_5 loading rate is usually sufficient for predominantly domestic wastes.
- Consider peak hourly BOD_5 for significant industrial waste loadings.
- Consider seasonal variations in domestic and/or industrial BOD_5 loading rates.
- Consider hydraulic detention time for short detention treatment systems (high-rate processes).

Aeration (with nitrification)

- Average BOD_5 loading rate is usually sufficient for predominantly domestic wastes.
- Consider peak hourly BOD_5 for significant industrial waste loadings.
- Peak daily flow and peak daily ammonia (Total Kjeldahl Nitrogen (TKN) for extended aeration) loading rates.
- Daily or seasonal variations in BOD_5, ammonia, (TKN with extended aeration) and peak flowrates should also be taken into consideration.

Secondary Sedimentation

- Peak hourly flow (peak daily flow may be considered in systems with adequate flow equalization).
- Peak solids loading rate.

Sludge Return

- Capacity requirements will vary with the treatment system (see Section 7.1.4).

Disinfection Systems

- Peak hourly flow.
- Consider peak instantaneous flows from batch processes.

Effluent Filtration

- Peak hourly flow.
- Peak flowrate.
- Peak solids loading rate.

Outfall Sewer

- Peak instantaneous flow, considering bypasses.

Sludge Treatment (Digestion, Thickening, Dewatering, Incineration, etc.)

- Average loading rates (hydraulic, Total Solids (TS), volatile solids) unless sustained peaks are of significance to the individual treatment process.

Effluent Retention Pond

- Average daily flow for the anticipated low flow period (in a low flow receiving stream).

5.4.4 Conduits

All piping and channels should be designed to carry the maximum expected flows. The incoming sewer should be designed for unrestricted flow. Bottom corners of the channels must be filleted. Conduits should be designed to avoid creation of pockets and corners where solids can accumulate.

Suitable gates should be placed in channels to seal off unused sections which might accumulate solids. The use of shear gates or stop planks is permitted where they can be used in place of gate valves or sluice gates. Non-corrodible materials should be used for these control gates.

5.4.5 Arrangement of Units

Component parts of the plant should be arranged for greatest operating and maintenance convenience, flexibility, continuity of optimum effluent quality for water quality protection, economy of function, and ease of installation of future units.

Where parallel units are provided, a central collection and distribution point including proportional flow splitting should be provided for the wastewater flow before each unit operation. Exceptions to this central collection and distribution point requirement may be made on a case-by-case basis when the design incorporates more than one unit process in the same physical structure.

5.4.6 Component Back-Up Requirements

The components of WWTPs should be designed in such a way that equipment breakdown and normal maintenance operations can be accommodated without causing serious deterioration of effluent quality.

To achieve this, critical treatment processes should be provided in multiple units so that with the larger unit out of operation, the hydraulic capacity (not necessarily the design rated capacity) of the remaining units should be sufficient to handle the peak wastewater flow. There should also be sufficient flexibility in capability of operation so that the normal flow into a unit out of operation can be distributed to all the remaining units. Similarly, it should be possible to distribute the flow of all the units in the treatment process downstream of the affected process. In addition, where feasible, it should be possible to operate the sections of treatment plants as completely separate process trains to allow full-scale loading tests to be carried out.

5.4.7 Flow Division Control

Flow division control facilities should be provided as necessary to ensure organic and hydraulic loading control to plant process units and should be designed for easy operator access, change, observation, and maintenance. The use of upflow division boxes equipped with adjustable sharp-crested weirs or similar devices is recommended. The use of valves for flow splitting is not acceptable. Appropriate flow measurement facilities should be incorporated in the flow division control design.

5.4.8 Plant Hydraulic Gradient

The hydraulic gradient of all gravity flow and pumped waste streams within the WWTP, including by-pass channels, should be prepared to ensure that adequate provision has been made for all head losses.

In calculating the hydraulic gradient, changes in head caused by all factors should be considered, including the following:

- Head losses due to channel and pipe wall friction.
- Head losses due to sudden enlargement or sudden contraction in flow cross section.
- Head losses due to sudden changes in direction, such as at bends, elbows, Wye branches, and Tees.
- Head losses due to sudden changes in slope or drops.
- Head losses due to obstructions in conduits.
- Head required to allow flow over weirs, through flumes, orifices, and other measuring, controlling, or flow division devices.
- Head losses caused by flow through comminutors, bar screens, tankage, filters, and other treatment units, considering clogging that may arise through normal operation (e.g., buildup of material on intermittently cleaned screens).
- Head losses caused by air entrainment or air binding.
- Head losses incurred due to flow splitting along the side of a channel.
- Head increases caused by pumping.
- Head allowances for expansion requirements and/or process changes.
- Head allowances due to maximum water levels in receiving waters (including climate change considerations such as sea level rise, potential increases in storm surges, and changes in flood elevations, etc.).

5.5 Plant Details

5.5.1 Installation of Mechanical Equipment

The specifications should be written so that the installation and initial operation of major items of mechanical equipment will be inspected and approved by a representative of the manufacturer.

5.5.2 Unit Bypasses

5.5.2.1 Removal from Service General

Properly located and arranged bypass structures and piping should be provided so that each unit of the plant can be removed from service independently. The bypass design should facilitate plant operation during unit maintenance and emergency repair to minimize deterioration of effluent quality and ensure rapid process recovery upon return to normal operational mode.

Bypassing may be accomplished using duplicate or multiple treatment units in any stage if the design peak instantaneous flow can be handled hydraulically with the largest unit out of service.

The actuation of all bypasses should require manual action by operating personnel. All power-actuated bypasses should be designed to permit manual operation in the event of power failure and should be designed so that the valve will fail in last position, upon failure of the power actuator.

A fixed high water level bypass overflow should be provided in addition to a manual or power actuated bypass.

5.5.2.2 Unit Bypass During Construction

Unit bypassing during construction should be in accordance with the plan for the method and level of treatment (including sludge processing, storage, and disposal) to be achieved during construction, which should be developed and submitted to the Regulatory Authority for review and approval and, where required, to Environment and Climate Change Canada for a temporary bypass authorization under the WSER.

5.5.2.3 Unit Dewatering, Flotation Protection, & Plugging

Means such as drains or sumps should be provided to completely dewater each unit to an appropriate point in the process. Due consideration should be given to the possible need for hydrostatic pressure relief devices to prevent flotation of structures. Pipes subject to plugging should be provided with means for mechanical cleaning or flushing.

5.5.3 Overflows

If wastewater entering the treatment plant must be pumped into the treatment units, an emergency overflow for the pumping station should be provided, if it is physically possible (see Section 3.3.2). The purpose of this overflow is to prevent basement flooding by back-ups in the sewer system in the event of pumping station failure. Wherever possible, this overflow should be routed through the treatment plant disinfection systems and plant outfall sewer. If this is not possible, chlorination and dechlorination of such overflows should be considered.

The overflow elevation and the method of activation should ensure that the maximum feasible storage of the wet well will be utilized before the controlled overflow takes place. The overflow facilities should be alarmed and should be equipped to indicate frequency and duration of overflows and provided with facilities to permit manual flow measurement. Automatic flow measurement and recording systems may be required in certain cases where effluent quality requirements dictate. Where the operator is signatory to a *Shellfish Conditional Area Management Plan (Management of shellfish harvesting in areas next to wastewater treatment plants* from Fisheries and Oceans Canada) notification and reporting requirements of the plan should be met. All overflows should be recorded and reported to the Regulatory Authorities.

5.5.4 Construction Materials

Due consideration should be given to the selection of materials which are to be used in wastewater treatment works because of the possible presence of hydrogen sulphide and other corrosive gases, greases, oils, and similar constituents frequently present in wastewater. This is particularly important in the selection of metals and paints. Dissimilar metals should be avoided to minimize galvanic action.

5.5.5 Painting

The use of paints containing lead or mercury should be avoided. To facilitate identification of piping, particularly in the large plants, it is suggested that the different lines be colour coded. The colour scheme presented in Table 5.1 is recommended for purposes of standardization.

Table 5.1: Recommended Colour Scheme for Standardization

Piping Type	Colour
Raw sludge line	Gray
Sludge recirculation suction line	Brown with yellow bands
Sludge draw off line	Brown with orange bands
Sludge recirculation discharge line	Brown
Digested sludge line	Black
Sludge gas line	Red
Natural gas line	Red
Non-potable water line	Purple
Potable water line	Blue
Fire main	Red
Chlorine line	Yellow
Sulfur dioxide	Yellow with red bands
Wastewater line	Gray
Compressed air line	Dark green
Process airline	Light green
Water lines for heating digesters or buildings	Blue with a 150 mm red band spaced 750 mm apart
Fuel oil/diesel	Red
Plumbing drains and vents	Black
Ferric chloride	Orange
Polymer	Unpainted PVC

The contents and direction of flow should be stencilled on the piping in a contrasting colour.

5.5.6 Operating Equipment

A complete outfit of tools, accessories, and spare parts necessary for the plant operators use should be provided.

A portable pump is desirable. Readily accessible storage space and work bench facilities should be provided, and consideration given to provision of a garage area which would also provide space for large equipment, maintenance, and repair.

5.5.7 Erosion Control During Construction

Effective site erosion control should be provided during construction as required by local jurisdiction. An approved erosion control plan is required before construction begins.

5.5.8 Grading & Landscaping

Upon completion of the plant, the ground should be graded and sodded or seeded. All-weather walkways should be provided for access to all units. Where possible, steep slopes should be avoided to prevent erosion. Surface water should not be permitted to drain into any unit. Particular care should be taken to protect trickling filter beds, sludge beds, and intermittent sand filters from storm water runoff. Provisions should be made for landscaping, particularly when a plant must be located near residential areas.

5.5.9 Cathodic Protection

Steel fabricated WWTPs should be equipped with cathodic protection for corrosion control as specified in Section 4.2.12.

5.6 Plant Outfalls

5.6.1 Discharge Impact Control

Outfall sewers should consist of a completely piped system conforming to the requirements of these Guidelines and should not discharge into any ditch or watercourse in which adequate assimilative capacity is not available. In assessing the available assimilative capacity, the proximity of other outfalls must be taken into consideration. The impacts of climate change (e.g., potential decreasing low flows and increased frequency of drought conditions) as it affects assimilative capacity should be considered.

Consideration should be given to limited or complete across-stream dispersion as needed to protect aquatic life movement and growth in the immediate reaches of the receiving water.

5.6.1.1 Submerged Outlet

The outfall sewer should have its outlet submerged if physically possible. Where greater depths are available, 1 m should be the minimum achieved depth of submergence. This should account for future water level changes, including climate change considerations (e.g., potential reductions in low water levels during drought conditions), if this is projected to reduce the depth of submergence.

5.6.1.2 Dispersion of Flow

Where conditions exist that a point discharge of effluent could have deleterious effects on the receiving body of water, consideration should be given to providing a means of effective submerged dispersion of the effluent into the receiving water.

5.6.2 Protection & Maintenance

The outfall sewer should be so constructed and protected against the effects of flood water, tides, ice, or other hazards as to reasonably insure its structural stability and freedom from stoppage. Hazards to navigation should be considered in designing outfall sewers.

5.6.3 Sampling Provisions

All outfalls should be designed so that a sample of the effluent can be obtained at a point after the final treatment process as well as downstream of any plant bypasses and before discharge to or mixing with the receiving water.

5.7 Essential Facilities

5.7.1 Emergency Power Facilities

5.7.1.1 General

All plants should be provided with an alternate source of electric power or pumping capability to allow continuity of operation during power failures, except as noted below. Refer to Section 4.3.5 for design details.

Methods of providing alternate sources include:

- The connection of at least two independent power sources such as substations able to supply power without interruption. A power line from each substation and separate routes are recommended and will be required unless documentation is received and approved by the Regulatory Authority verifying that a duplicate line is not necessary.
- Portable or in-place internal combustion engine equipment which will generate electrical or mechanical energy.
- Portable pumping equipment when only emergency pumping is required.

The generator should be located above flood elevations, including climate change considerations (refer to Chapter 2 for further guidance). Provisions for stand-by power are important to reduce risk of interruptions to operations and provide resiliency to climate change (e.g., potential increased risk of power outages from projected increases in the frequency and intensity of weather events).

5.7.1.2 Power for Aeration

Standby generating capacity is required for aeration equipment used in the activated sludge process to provide minimum aeration. Full power generating capacity may be required by the Regulatory Authority for waste discharges to certain critical stream segments such as upstream of bathing beaches, upstream of a public water supply intake, or other similar situations.

5.7.1.3 Power for Disinfection

Continuous disinfection, where required, should be provided during all power outages. Continuous dechlorination is required for systems that dechlorinate.

5.7.1.4 Power for Data Loggers

Computers configured to log data should be supplied with an Uninterruptable Power Supply (UPS). Each UPS should monitor its own battery condition and issue alarms on low battery. Uninterruptable power supplies configured to supply computers should cause the computer to save all open files and data logging files, without overwriting existing files, at the time of primary power failure and again when a low battery condition occurs.

5.7.2 Water Supply

5.7.2.1 General

An adequate supply of potable water under pressure should be provided for use in the laboratory, chlorination equipment and general cleanliness around the plant. The chemical quality should be checked for suitability for its intended uses such as heat exchangers, chlorinators, etc.

No piping or other connections should exist in any part of the treatment works, which, under any conditions, might cause the contamination of a potable water supply. If a potable water supply is brought to the plant, it should be protected with a suitable backflow prevention device as determined by CSA standards.

5.7.2.2 Direct Connections
Potable water from a municipal or separate supply may be used directly at points above grade for the following hot and cold supplies:
- Sink.
- Water closet.
- Laboratory sink.
- Shower.
- Drinking fountain.
- Eye wash fountain.
- Safety shower.

Hot water for any of the above units should not be taken directly from a boiler used for supplying hot water to a sludge heat exchanger or digester heating coils.

5.7.2.3 Indirect Connections
Where a potable water supply is to be used for any purpose in a plant other than those listed in Section 5.7.2.2, a break tank, pressure pump, and pressure tank should be provided. Water should be discharged to the break tank through an air-gap at least 150 mm above the maximum flood line or the spill line of the tank, whichever is higher.

A sign should be permanently posted at every hose bib, faucet or sill cock located on the water system beyond the break tank to indicate that the water is not safe for drinking.

Consideration will also be given to backflow prevention devices consisting of a system of check valves and relief valves which provide protection against backflow (reduced pressure zone assemblies).

5.7.2.4 Separate Potable Water Supply
Where it is not possible to provide potable water from a public water supply, a separate well may be provided as long as sufficient pressure is available. Location and construction of the well should comply with requirements of the Regulatory Authority. Requirements governing the use of the supply are those contained in Sections 5.7.2.2 and 5.7.2.3.

5.7.2.5 Separate Non-Potable Water Supply
Where a separate non-potable water supply is to be provided, a break tank will not be necessary, but all sill cocks and hose bibs should be posted with a permanent sign indicating the water is not safe for drinking.

5.7.3 Sanitary Facilities
Toilet, shower, lavatory, and locker facilities should be provided in sufficient numbers and convenient locations to serve the expected plant personnel.

5.7.4 Floor Slope
Floor surfaces should be sloped adequately to a point of drainage.

5.7.5 Stairways

Stairways should be installed in lieu of ladders for access to units requiring routine inspection and maintenance, such as:

- Digesters.
- Trickling filters.
- Aeration tanks.
- Clarifiers.
- Tertiary filters, etc.

Spiral or winding stairs are permitted only for secondary access where dual means of egress are provided.

Stairways should have slopes between 30 to 35° from the horizontal to facilitate carrying samples, tools, etc. Each tread and riser should be of uniform dimension in each flight. Minimum tread run should not be less than 200 mm. The sum of the tread run and riser should not be less than 430 mm and no more than 460 mm. A stairway should not have more than a 3.0 m continuous rise without a platform. The *National Building Code of Canada* from the National Research Council of Canada supersedes these clauses if there are contradictions between the two.

5.7.6 Wastewater Flow Measurement

Facilities for measuring and recording all wastewater flows through the treatment works should be provided and should meet federal WSER and provincial requirements where applicable. All plant and process unit bypasses should also be equipped with flow measuring devices, such that hydraulic balances around each treatment process unit and the total plant are possible. Flow measuring devices should be located so that the flows measured are meaningful and recordable.

5.7.6.1 Location

Flow measurement equipment should be provided to measure the following flows:

- Plant influent or effluent flow (if influent flow is significantly different from effluent flow, both should be measured). This would apply for installations such as stabilization ponds, and plants with excess flow storage or flow equalization).
- Bypass flow around WWTP.
- Other flows required to be monitored under the provisions of the Approval/License/Permit to Operate.
- Other flows such as Return Activated Sludge (RAS), Waste Activated Sludge (WAS), recirculation, and recycle are required for plant operational control.

5.7.6.2 Equipment

Indicating, totalizing, and recording flow measurement devices should be provided for all plants in accordance with WSER requirements, where applicable. All flow measurement equipment must be sized to function effectively over the full range of flows expected and should be protected against freezing. For plants where WSER does not apply, flow measuring devices that can be read and recorded manually should be provided as a minimum, although indicating, totalizing, and recording flow measurement devices are recommended. Potable water flow measurement may be acceptable in systems serving one building where I&I is well controlled, upon approval from Regulatory Authority.

See Section 4.2.3.5 for the requirements concerning electrical systems and components located in enclosed or partially enclosed spaces where hazardous concentrations of flammable gases or vapors may be present.

5.7.6.3 Hydraulic Conditions

Flow measurement equipment including approach and discharge conduit configuration and critical control elevations should be designed to ensure that the required hydraulic conditions necessary for accurate measurement are provided. Turbulence, eddy currents, air entrainment or any other aspect that upsets the normal hydraulic conditions that are necessary for accurate flow measurement should be avoided.

5.7.7 Sampling Equipment

Effluent composite sampling equipment should be provided at all plants with an ADF of 2,500 m^3/d or greater and at other facilities where it is necessary to meet Approval/License/Permit to Operate requirements or monitoring requirements under WSER legislation.

Composite sampling equipment should also be provided as needed for influent sampling and for monitoring plant operations. The influent sampling point should be located prior to any process return flows.

Refer to Section 4.2.3.5 for the requirements concerning electrical systems and components located in enclosed or partially enclosed spaces where hazardous concentrations of flammable gases or vapors may be present. This paragraph should be considered in the design and location of influent composite sampling equipment, as well as effluent composite sampling equipment if a bypass allows untreated wastewater to reach the effluent sample point.

5.8 Safety

5.8.1 General

Adequate provisions should be made to effectively protect the operator and visitors from hazards. The provincial building code supersedes the following clauses if there are contradictions between the two. The following should be provided to fulfil the particular needs of each plant:

- Closure of the plant site with a fence and signs designed to discourage the entrance of unauthorized persons and animals.
- Handrails with toe-boards where appropriate.
- Guards around tanks, trenches, pits, stairwells, and other hazardous structures where the top of the wall is less than 1,070 mm above the surrounding ground level.
- Gratings over appropriate areas of treatment units where access for maintenance is required, with consideration of temporary barriers for times when grating must be removed.
- First aid equipment.
- “No Smoking” signs in hazardous areas.
- Protective clothing and equipment as needed, such as self-contained breathing apparatus, gas detection equipment, goggles, gloves, hard hats, safety harnesses, hearing protectors, etc.
- Portable blowers and sufficient hose.
- Portable lighting equipment complying with the latest edition of *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations* and provincial electrical code requirements.

- Proper exit signage and permanently installed emergency lighting in accordance with the latest edition of the *National Building Code of Canada* from the National Research Council of Canada, *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations*, and *NFPA 101 Life Safety Code*.
- Gas detectors listed and labeled for use in Class I, Division 1, Group D locations.
- Appropriately placed warning signs for slippery areas, non-potable water fixtures, low head clearance areas, open service manholes, hazardous chemical storage areas, flammable fuel storage areas, high noise areas, etc.
- Adequate ventilation in pump station areas in accordance with Section 4.2.7.
- Provisions for local lockout on stop motor controls.
- Provisions for confined space entry and laboratory safety in accordance with Regulatory Authority requirements.
- Adequate vector control.

Consideration should also be given during design to accommodate future O&M and safety concerns. This could include:
- Anchor points.
- Davit holders.
- Clearance for large equipment replacement.

5.8.2 Hazardous Chemical Handling

Reference should be made to the latest edition of the *Transportation of Dangerous Goods Act* from Transport Canada and applicable provincial acts or regulations.

5.8.2.1 Contaminant Materials

The materials utilized for storage, piping, valves, pumping, metering, splash guards, etc., should be specially selected considering the physical and chemical characteristics of each hazardous or corrosive chemical.

5.8.2.2 Chemical Storage & Handling Areas

Structures, rooms, and areas accommodating chemical storage and feed equipment should be arranged to provide convenient access for chemical deliveries, equipment servicing, repair, and observation of operation. It is recommended that wherever possible the storage area be separated from the main plant, and that segregated storage be provided for each chemical. Where two or more chemicals could react with undesirable effects, the drainage piping (if provided) from the separate chemical handling areas should not be interconnected. For dangerous materials such as gaseous chlorine, floor drains in the storage and scale rooms should be omitted entirely, with the floors sloped towards the doors.

5.8.2.3 Secondary Containment

Chemical storage areas should be enclosed in dykes or curbs which will contain 110% of the stored volume until it can be safely transferred to alternate storage or released to the wastewater at controlled rates which will not damage facilities, inhibit the treatment processes, or contribute to stream pollution. Chemical containment pallets may be used for smaller chemical storage such as drums and totes in lieu of curbs and dykes if sufficient storage volume can be provided. Liquid polymer should be similarly contained to reduce areas with slippery floors, especially to protect travel ways. Non-slip floor surfaces are desirable in polymer handling areas.

5.8.2.4 Underground Storage

Underground storage and piping facilities for fuels or for chemicals such as alum or ferric chloride, should be constructed in accordance with applicable provincial and federal regulations on underground storage tanks for both fuels and hazardous materials.

5.8.2.5 Liquified Gas Chemicals

Areas intended for storage and handling of chlorine, sulfur dioxide, and other hazardous gases should be properly designed and isolated. Gas detection kits, alarms, controls, safety devices, and emergency repair kits should be provided.

5.8.2.6 Splash Guards

All pumps or feeders for hazardous or corrosive chemicals should have guards which will effectively prevent spray of chemicals into space occupied by personnel. The splash guards are in addition to guards to prevent injury from moving or rotating machinery parts.

5.8.2.7 Piping, Labelling, Coupling Guards, & Location

All piping containing or transporting corrosive or hazardous chemicals should be identified with labels every 3 m and with at least two labels in each room, closet, or pipe chase. Colour coding may also be used but is not an adequate substitute for labelling.

All connections (flanged or other type), except adjacent to storage or feeder areas, should have guards which will direct any leakage away from space occupied by personnel. Pipes containing hazardous or corrosive chemicals should not be located above shoulder level except where continuous drip collection trays and coupling guards will eliminate chemical spray or dripping onto personnel.

5.8.2.8 Protective Clothing or Equipment

The following items of protective clothing or equipment should be available and utilized for all operations or procedures where their use will minimize injury hazard to personnel:

- Self-contained air supply system recommended for protection against chlorine.
- Chemical worker’s goggles or other suitable goggles (safety glasses are insufficient).
- Face masks or shields for use over goggles.
- Dust mask or respirator to protect the lungs in dry chemical areas, as appropriate.
- Rubber gloves.
- Rubber aprons with leg straps.
- Rubber boots (leather and wool clothing should be avoided near caustics).
- Safety harness and line.

5.8.2.9 Warning Systems & Signs

Facilities should be provided for automatic shut-down of pumps and sounding of alarms when failure occurs in a pressurized chemical discharge line.

Warning signs requiring use of goggles should be located near chemical unloading stations, pumps, and other points of frequent hazard.

5.8.2.10 Dust Collection

Dust collection equipment should be provided to protect personnel from dusts injurious to the lungs or skin and to prevent polymer dust from settling on walkways. The latter is to minimize slick floors which result when a polymer covered floor becomes wet.

5.8.2.11 Eyewash Foundations & Safety Showers

Eyewash fountains and safety showers utilizing an approved flushing fluid (potable water, preserved water, preserved buffered saline solution or other medically acceptable solutions) should be provided on each floor level or work location involving hazardous or corrosive chemical storage, mixing (or slaking), pumping, metering, or transportation unloading. These facilities should be as close as practical to work location and no more than 7.5 m from points of chemical exposure and should be fully operable during all weather conditions.

Each eyewash fountain should be supplied with flushing fluid at a tepid temperature (16°C to 38°C) suitable to provide at least 15 minutes of continuous irrigation of the eyes at a rate of 1.5 L/minute, or as required by the Safety Data Sheets (SDSs) for the types of hazardous chemical being handled at the plant. Each emergency shower should be capable of discharging at least 76 L/minute of flushing fluid at a tepid temperature (16°C to 38°C) to provide at least 15 minutes of continuous flushing and should be at pressures of 210 to 345 kPa (30 to 50 psi). Anti-scalding devices should be provided as required to maintain a relatively constant temperature. Refer to the latest edition of *ANSI/ISEA Z358.1 American National Standard for Emergency Eyewash and Shower Equipment* from the American National Standards Institute (ANSI) for more information.

5.8.3 Hazardous Chemical Container Identification

The identification and hazard warning data included on shipping containers, when received, should appear on all containers (regardless of size or type) used to store, carry, or use a hazardous substance. Wastewater and sludge sample containers should be adequately labeled according to Workplace Hazardous Materials Information System (WHMIS).

5.9 Laboratory

5.9.1 General

All treatment plants should include a laboratory for making the necessary analytical determinations and operating control tests, except for plants utilizing only processes not requiring laboratory testing for plant control where satisfactory off-site laboratory provisions are made to meet the permit monitoring requirements. For plants where a fully equipped laboratory is not required, the requirements for utilities, fume hoods, etc., may be reduced. The laboratory should have sufficient size, bench space, equipment, and supplies to perform all self-monitoring analytical work required by discharge permits, and to perform the process control tests necessary for good management of each treatment process included in the design.

The facilities and supplies necessary to perform analytical work to support industrial waste control programs will normally be included in the same laboratory. The laboratory arrangement should be sufficiently flexible to allow future expansion should more analytical work be needed. Laboratory instrumentation and size should reflect treatment plant size, staffing requirements, process complexity, and applicable certification requirements. Experience and training of plant operators should also be assessed when determining treatment plant laboratory needs.

Consult the Regulatory Authority to see whether accredited laboratories are required to perform all regulatory testing.

Treatment plant laboratory needs may be divided into the following three general categories:
- Plants performing only basic operational testing (this typically includes pH, temperature, and Dissolved Oxygen (DO)).
- Plants performing intermediate laboratory testing (more complex operational and permit laboratory tests including BOD, SS, and bacterial analysis).
- Plants performing advanced laboratory testing (more complex operational, permit, industrial pretreatment, and multiple plant laboratory testing).

Expected minimum laboratory needs for these three plant classifications are outlined in this section, however, in specific cases, laboratory needs may have to be modified or increased due to industrial monitoring needs or special process control requirements.

5.9.2 Category I: Basic Operational Testing

5.9.2.1 Location & Space

A floor area up to 14 m^2 should be adequate. It is recommended that this be at the treatment site. Another location in the community utilizing space in an existing structure owned by the System Owner may be acceptable.

5.9.2.2 Design & Materials

The plant should provide for electricity, water, heat, sufficient storage space, a sink, and a bench top. The lab components need not be of industrial grade materials. Laboratory equipment and glassware should be of types recommended by *Standard Methods for the Examination of Water and Wastewater* from the American Public Health Association, the American Water Works Association, and the Water Environment Federation as well as the Regulatory Authority.

5.9.3 Category II: Intermediate Laboratory Testing

5.9.3.1 Location & Space

The laboratory size should be based on providing adequate room for the equipment to be used. In general, the laboratories for this category of plant should provide a minimum of approximately 28 m^2 of floor space. The laboratory should be located at the treatment site on ground level. It should be isolated away from vibrating, noisy, high-temperature machinery or equipment which might have adverse effects on the performance of laboratory staff or instruments.

5.9.3.2 Floors

Floor surfaces should be fire resistant, and highly resistant to acids, alkalis, solvents, and salts.

5.9.3.3 Cabinets & Bench Tops

Laboratories in this category usually perform both the permit testing and operational control monitoring utilizing “acids” and “bases” in small quantities, such that laboratory grade metal cabinets and shelves are not mandatory. The cabinets and shelves selected may be of wood or other durable materials. Bench tops should be of acid resistant laboratory grade materials for protection of the non-acid proof cabinets. Glass doors on wall-hung cabinets are not required. One or more cupboard style base cabinets should be provided. Cabinets with drawers should have stops to prevent accidental removal. Cabinets for Category II laboratories are not required to have gas, air, vacuum, and electrical service fixtures. Built-in shelves should be adjustable.

5.9.3.4 Fume Hoods, Sinks, & Ventilation

5.9.3.4.1 Fume Hoods

Fume hoods should be provided for laboratories in which required analytical works results in the production of noxious fumes.

5.9.3.4.2 Sinks

A laboratory grade sink and drain trap should be provided.

5.9.3.4.3 Ventilation

Laboratories should be air conditioned. In addition, separate exhaust ventilation should be provided.

5.9.3.5 Balance & Table

An analytical balance of the automated digital readout, single pan 0.1 mg sensitivity type should be provided. A heavy special-design balance table which will minimize vibration of the balance is recommended. It should be located as far as possible from windows, doors, or other sources of drafts or air movements, so as to minimize undesirable impacts from these sources upon the balance.

5.9.3.6 Equipment, Supplies, & Reagents

The laboratory should be provided with all of the equipment, supplies, and reagents that are needed to carry out all of the plant's analytical testing requirements. If any required analytical testing produces malodorous or noxious fumes, the engineer should verify that the in-house analysis is more cost-effective than use of an independent off-site laboratory. Composite samples may be required to satisfy permit sampling requirements. The Approval/License/Permit to Operate, process control, and industrial waste monitoring requirements should be considered when specifying equipment needs. References such as *Standard Methods for the Examination of Water and Wastewater* from the American Public Health Association, the American Water Works Association, and the Water Environment Federation as well as the *Clean Water Act Analytical Methods* from the United States Environment Protection Agency should be consulted prior to specifying equipment items.

5.9.3.7 Utilities

5.9.3.7.1 Power Supply

Consideration should be given to providing line voltage regulation for power supplied to laboratories using delicate instruments.

5.9.3.7.2 Laboratory Water

Reagent water of a purity suitable for analytical requirements should be supplied to the laboratory. In general, reagent water prepared using an all glass distillation system is adequate, however, some analyses require deionization of the distilled water. Consideration should be given to softening the feed water to the still.

5.9.3.8 Safety

5.9.3.8.1 Equipment

Laboratories should provide as a minimum the following:

- First aid equipment.
- Protective clothing including goggles, gloves, lab aprons, etc.
- A fire extinguisher.

5.9.3.8.2 Eyewash Fountains & Safety Showers

Eyewash fountains and safety showers should be provided as per Section 5.8.2.11.

5.9.4 Category III: Advanced Laboratory Testing

5.9.4.1 Location & Space

The laboratory should be located at the treatment site on ground level, with environmental control as an important consideration. It should be located away from vibrating, noisy, high temperature machinery or equipment which might have adverse effects on the performance of laboratory staff or instruments.

The laboratory facility needs for Category III plants should be described in the engineering Design Report or facilities plan. The laboratory floor space and facility layout should be based on an evaluation of the complexity, volume, and variety of sample analyses expected during the design life of the plant including testing for process control, industrial pre-treatment control, user charge monitoring, and monitoring requirements.

Consideration should be given to the necessity to provide separate (and possibly isolated) areas for some special laboratory equipment, glassware, and chemical storage. The analytical and sample storage areas should be isolated from all potential sources of contamination. It is recommended that the organic chemical facilities be isolated from other facilities. Adequate security should be provided for sample storage areas. Provisions for the proper storage and disposal of chemical wastes should be made. At large plants, office and administrative space needs should be considered.

For less complicated laboratory needs bench-top working surface should occupy at least 35% of the total laboratory floor space. Additional floor and bench space should be provided to facilitate the performance of analysis of industrial wastes, as required by the discharge permit and the System Owner's industrial waste pre-treatment program. Ceiling height should be adequate to allow for the installation of wall mounted water stills, deionizers, distillation racks, hoods, and other equipment with extended height requirements.

5.9.4.2 Floors & Doors

5.9.4.2.1 Floors

Floor surfaces should be fire resistant, and highly resistant to acids, alkalis, solvents, and salts. Floor surfaces should be a single color for ease of locating dropped items. The structural floor should be concrete with no basement.

5.9.4.2.2 Doors

Two exit doors should be located to permit straight egress from the laboratory, preferably at least one to the outside of the building. Doors should have a minimum width of 915 mm and should open in the direction of exit traffic. Panic hardware should be used. They should have large glass windows for easy visibility of approaching or departing personnel. Automatic door closers should be installed; swinging doors should not be used.

Flush hardware should be provided on doors if cart traffic is anticipated. Kick plates are also recommended.

5.9.4.3 Cabinets & Bench Tops

5.9.4.3.1 Cabinets

Wall-hung cabinets are recommended for dust-free storage of instruments and glassware. Units with sliding glass doors are recommended. A reasonable proportion of cupboard style base cabinets and drawer units should be provided. All cabinet shelving should be acid resistant and adjustable.

Drawers should slide out so that entire contents are easily visible. They should be provided with rubber bumpers and stops to prevent accidental removal. Drawers should be supported on ball bearings or nylon rollers which pull easily in adjustable steel channels. All metal drawer fronts should be double-wall construction.

The laboratory furniture should be supplied with adequate water, gas, air, and vacuum service fixtures, traps, strainers, plugs, and tailpieces, and electrical service fixtures.

5.9.4.3.2 Benchtops

Bench tops should be constructed of materials resistant to attacks from normally used laboratory reagents. Generally, bench-top height should be 900 mm, however, areas to be used exclusively for sit-down type operations should be 760 mm high and include kneehole space. Twenty-five (25) mm overhangs and drip grooves should be provided to keep liquid spills from running along the face of the cabinet. Tops should be furnished in large sections, 32 mm thick. They should be field-jointed into a continuous surface with acid, alkali, and solvent-resistant cements which are at least as strong as the material of which the top is made.

5.9.4.4 Hoods

5.9.4.4.1 General

Fume hoods to promote safety should be provided for laboratories where required analytical work results in the production of noxious fumes. Canopy hoods overheat releasing equipment should be provided.

5.9.4.4.2 Fume Hoods

Location

Fume hoods should be located where air disturbance at the face of the hood is minimal. Air disturbance may be created by:

- Persons walking past the hood.
- Heating, ventilating, or air-conditioning systems.
- By drafts from opening or closing a door, etc.

Safety factors should be considered in locating a hood. If a hood is situated near a doorway, a secondary means or egress must be provided. Bench surfaces should be available next to the hood so that chemicals need not be carried long distances.

Design and Material
The selection, design, and materials of construction of fume hoods and their appropriate safety alarms should be made by considering the variety of analytical work to be performed. The characteristics of the fumes, chemicals, gases, or vapors that will or may be released by the activities therein should be considered. Special design and construction is necessary if perchloric acid use is anticipated. Consideration should be given to providing more than one fume hood to minimize potentially hazardous conditions throughout the laboratory. Air intake should be balanced against all exhaust ventilation to maintain an overall positive pressure relative to atmospheric in the laboratory.

Fume hoods are not appropriate for operation of heat-releasing equipment that does not contribute to hazards, unless they are provided in addition to those needed to perform hazardous tasks.

Mixtures
One sink should be provided inside each fume hood. A cup sink is usually adequate.

All switches, electrical outlets, and utility and baffle adjustment handles should be located outside the hood. Light fixtures should be explosion-proof.

Exhaust
24-hour continuous exhaust capability should be provided. Exhaust fans should be explosion-proof. Exhaust velocities should be checked when fume hoods are installed.

5.9.4.4.3 Canopy Hoods
Canopy hoods should be installed over the bench-top areas where hot plate, steam bath, or other heating equipment or heat-releasing instruments are used. The canopy should be constructed of heat and corrosion resistant material.

5.9.4.5 Sinks, Ventilation, & Lighting
5.9.4.5.1 Sinks
The laboratory should have a minimum of two sinks (not including cup sinks). At least one of them should be a double-well sink with drain boards. A sink dedicated to hand washing should be provided. Additional sinks should be provided in separate work areas as needed and identified for the use intended.

Sinks and traps should be made of epoxy resin or plastic materials highly resistant to acids, alkalies, solvents, and salts, and should be abrasion and heat resistant, non-absorbent, and lightweight. Traps should be made of glass, plastic, or lead when appropriate and easily accessible for cleaning. Waste openings should be located toward the back so that a standing overflow will not interfere.

5.9.4.5.2 Ventilation
Laboratories should be separately air conditioned, with external air supplied for 100% make-up volume, also accounting for exhaust air from installed fume hoods. In addition, separate exhaust ventilation should be provided. Ventilation outlet locations should be remote from ventilation inlets. Consideration should be given to providing dehumidifiers. Air intake should be balanced against all supply air that is exhausted to maintain an overall positive pressure in the laboratory relative to atmospheric and other pressurized areas of the building which could be the source of airborne contaminants.

5.9.4.5.3 Lighting
Good lighting that is free from shadows must be provided throughout the laboratory for reading dials, meniscuses, etc.

5.9.4.6 Balance & Table
An analytical balance of the automatic, digital readout, single pan, 0.1 mg sensitivity type should be provided. A heavy special-design balance table which will minimize vibration of the balance is needed. It should be located as far as practical from windows, doors, or other sources of drafts or air movements, so as to minimize undesirable impacts from these sources upon the balance.

5.9.4.6.1 Microscope
A binocular or trinocular microscope with a 20-watt halogen light source, phase contrast condenser, mechanical stage, 10x, 40x, and 100x phase contrast objectives, wastewater reticule eyepiece and centering telescope is recommended for process control at activated sludge plants.

5.9.4.7 Equipment, Supplies, & Reagents
The laboratory should be provided with all of the equipment, supplies, and reagents that are needed to carry out all of the plant’s analytical testing requirements. Composite samplers may be required to satisfy permit sampling requirements. The Approval/License/Permit to Operate, process control, and industrial waste monitoring requirements should be considered when specifying equipment needs. Reference such as *Standard Methods for the Examination of Water and Wastewater* from the American Public Health Association, the American Water Works Association, and the Water Environment Federation as well as the *Clean Water Act Analytical Methods* from the United States Environment Protection Agency should be consulted prior to specifying equipment items.

5.9.4.8 Utilities & Services
5.9.4.8.1 Power Supply
Consideration should be given to providing line voltage regulation for power supplied to laboratories using delicate instruments.

5.9.4.8.2 Laboratory Water
Reagent water of a purity suitable for analytical requirements should be supplied to the laboratory. In general, reagent water prepared using an all glass distillation system is adequate, however, some analyses require deionization of the distilled water. Consideration should be given to softening water to the still.

5.9.4.8.3 Gas & Vacuum
Natural or liquefied petroleum gas should be supplied to the laboratory. Digester gas should not be used.

An adequately sized line source of vacuum should be provided with outlets available throughout the laboratory.

5.9.4.9 Safety

5.9.4.9.1 Equipment

Laboratories should provide the following:

- First aid equipment, protective clothing, and equipment such as:
  - Goggles.
  - Safety glasses.
  - Full face shields.
  - Gloves.
  - Fire extinguishers.
  - Chemical spill kits, etc.
- Posting of "No Smoking" signs in hazardous areas.
- Appropriately placed warning signs for the following:
  - Slippery areas.
  - Non-potable water fixtures.
  - Hazardous chemical storage areas.
  - Flammable fuel storage areas, etc.

5.9.4.9.2 Eyewash Fountains & Safety Showers

Eyewash fountains and safety should be provided as per Section 5.8.2.11. Adequate training should be provided to the plant O&M staff.

5.10 References

Canadian Council of Minister of the Environment, *Canadian Environmental Quality Guidelines*, 2005. <https://ccme.ca/en/summary-table>.

Canadian Council of Ministers of the Environment, "Development of a Canada-wide Strategy for the Management of Municipal Wastewater Effluent, Discussion Document" *Overview of Legislation, Regulations, Guidelines and Policies for the Management of Municipal Wastewater Effluents (MWWE) in Canada*, May 2004.

Canadian Council of Ministers of the Environment, *Environmental Risk-Based Approaches for Managing Municipal Wastewater Effluent (MWWE)*, 2005. <https://canadacommons.ca/artifacts/1216626/environmental-risk-based-approaches-for-managing-municipal-wastewater-effluent-mwwe/1769728/>.

Canadian Council of the Ministers of the Environment. "Canadian Environmental Quality Guidelines for the Protection of Aquatic Life." *Guidance on the Site-Specific Application of Water Quality Guidelines in Canada: Procedures for Deriving Numerical Water Quality Objectives*. Winnipeg: Canadian Council of the Ministers of the Environment, 2003. <https://ccme.ca/en/res/guidance-on-the-site-specific-application-of-water-quality-guidelines-in-canada-en.pdf>.

Canadian Council of the Ministers of the Environment. *Technical Supplement 2. Canada-wide Strategy for the Management of Municipal Wastewater Effluent. Environmental Risk Management: Framework and Guidance*. Canadian Council of Ministers of the Environment, June 2008. <https://ccme.ca/en/res/mwwe_techsupp_2_e.pdf>.

Canadian Institute on Pollution Control, *Manual on Wastewater Sampling Practice*, 1972.

Canadian Standards Association. *S900.1-2018 Climate Change Adaptation for Wastewater Treatment Plants*. CSA, 2018.

CSA Group. *Canadian Electrical Code C22.1:21*. CSA, 2021. Retrieved from <https://www.csagroup.org/store/product/C22.1-18/>.

Eckenfelder, W. Wesley, *Water Quality Engineering for Practicing Engineers*, Barnes & Noble, January 1970.

Government of Canada Justice Laws. *Wastewater Systems Effluent Regulations* (SOR/2012-139). Government of Canada, 2022. <https://laws-lois.justice.gc.ca/eng/regulations/sor-2012-139/fulltext.html>.

Government of Newfoundland and Labrador, Department of Environment and Conservation, Water Resources Management Division, *Guidelines for the Design, Construction and Operation of Water and Sewerage Systems*, December 2005. <https://www.gov.nl.ca/ecc/files/waterres-waste-groundwater-cover.pdf>.

International Safety Equipment Association. *ANSI/ISEA Z358.1-2014 American National Standard for Emergency Eyewash and Shower Equipment*. Arlington: International Safety Equipment Association, 2014. <https://webstore.ansi.org/Standards/ISEA/ANSIISEAZ3582014?source=blog>.

IOWA Department of Natural Resources, *Wastewater Facilities Design Standards* <https://www.iowadnr.gov/environmental-protection/water-quality/wastewater-construction/design-standards>.

National Fire Protection Association. *NFPA 101 Life Safety Code*. National Fire Protection Association (NFPA), April 2021. <https://www.nfpa.org/codes-and-standards/all-codes-and-standards/list-of-codes-and-standards/detail?code=101>.

National Research Council of Canada. "National Building Code of Canada." *Canadian Commission on Building and Fire Codes*. Government of Canada, 2015. <https://nrc.canada.ca/en/certifications-evaluations-standards/codes-canada/codes-canada-publications/national-building-code-canada-2015>.

New England Interstate Water Pollution Control Commission, *TR – 16, Guides for the Design of Wastewater Treatment Works*, 1998.

New York State Department of Environmental Conservation, *Manual of Instructions for Wastewater Treatment Plant Operators - Volume One and Two*, 1978.

Nova Scotia Department of the Environment, *Manual of Wastewater - Interim Guidelines - Collection and Treatment of Municipal Wastewater*, 1976.

Ontario Ministry of the Environment (Ontario MOE), *Water Management - Goals, Policies, Objectives and Implementation Procedures of the Ministry of Environment*, May 1984. Water Management: Goals, Policies, Objectives & Implementation Procedures of M.O.E. Revised 1984 (uoguelph.ca)

Ontario MOE, *Design Guidelines for Sewage Works*, May 2019. <https://www.ontario.ca/document/design-guidelines-sewage-works-0>.

Ontario MOE, *Stream Water Quality Assessment Procedures Manual*, March 1990.

Saskatchewan Environment and Public Safety, *Surface Water Quality Objectives*, June 2015. <https://www.wsask.ca/wp-content/uploads/2021/02/epb-356-surface-water-quality-objectives-interm-edition-june-2015.pdf>.

Saskatchewan Ministry of Environment, Water Security Agency, *Guidelines for Sewage Works Design*, April 2014. <https://www.acwwa.ca/resources/water-wastewater-guidelines/kmp/design-guidelines/194-saskatchewan-guidelines-for-sewage-works-design-2014/file>.

SENES Consultants Limited for Canadian Council of the Ministers of the Environment. *Technical Supplement 3. Canada-wide Strategy for the Management of Municipal Wastewater Effluent. Standard Method and Contracting Provisions for the Environmental Risk Assessment*. Canadian Council of the Ministers of the Environment, June 2008. <https://ccme.ca/en/res/mwwe_techsupp_3_e.pdf>.

Transport Canada. *Transportation of Dangerous Goods Act*. Government of Canada, 1992. <https://tc.canada.ca/en/dangerous-goods/transportation-dangerous-goods-act>.

U.S. Environmental Protection Agency (EPA). Manual: *Wastewater Treatment/Disposal for Small Communities*, EPA-625/1-92/005. Cincinnati, OH: Office of Research and Development, 1992.

U.S. Environmental Protection Agency, *Analysis of Operations and Maintenance Costs for Municipal Wastewater Treatment Systems*, 1978. <https://nepis.epa.gov/Exe/ZyPDF.cgi/9101CRB5.PDF?Dockey=9101CRB5.PDF>.

U.S. Environmental Protection Agency, *Considerations for Preparation of Operations and Maintenance Manuals*, 1974. https://nepis.epa.gov/Exe/ZyPDF.cgi/9100SFTU.PDF?Dockey=9100SFTU.PDF

U.S. Environmental Protection Agency, *Onsite Wastewater Treatment Systems, Technology Fact Sheet 11, Recirculating Sand/Media Filter*, February 2022. <https://www.epa.gov/sites/default/files/2015-06/documents/2004_07_07_septics_septic_2002_osdm_all.pdf>.

U.S. Environmental Protection Agency, *Process Design Manual - Wastewater Treatment Facilities for Sewered Small Communities*.

U.S. Environmental Protection Agency, *Technical Guidance Manual for Performing Waste Load Allocations - Book III Estuaries, Part 1 and 2*, March 1990. <https://www.epa.gov/sites/default/files/2019-12/documents/technical-guidance-wasteload-allocations-book3part1.pdf>.

U.S. Environmental Protection Agency, *Technical Guidance Manual for Providing Waste Load Allocations - Book IV Lakes and Impoundments - Chapter 1, 2, and 3*. https://www.epa.gov/sites/default/files/2019-12/documents/technical-guidance-wasteload-allocations-book3part4.pdf

U.S. Environmental Protection Agency, *Technical Guidance Manual for Providing Waste Load Allocations - Book VI Design Conditions - Chapter 1 - Stream Flow Design for Steady State Modelling*, September 1986. <https://permanent.fdlp.gov/websites/epagov/www.epa.gov/pdf-archive/wlabook6chapter1-1.pdf>.

U.S. Environmental Protection Agency, *Technical Guidance Manual for Providing Waste Load Allocations - Book II Streams and Rivers - Chapter 1, 2, and 3*, 1983. <https://nepis.epa.gov/Exe/ZyPDF.cgi/9101NBG1.PDF?Dockey=9101NBG1.PDF>., <https://nepis.epa.gov/Exe/ZyPDF.cgi/00001IUY.PDF?Dockey=00001IUY.PDF>., <https://permanent.fdlp.gov/websites/epagov/www.epa.gov/pdf-archive/wlabook2chapter3.pdf>.

U.S. Environmental Protection Agency, *Technical Guidance Manual for Providing Waste Load Allocations - Simplified Analytical Method for Determining NPDES Effluent Limitations for POTW's Discharging into Low-Flow Streams*, September 1980. <https://www.osti.gov/biblio/7007399-technical-guidance-manual-performing-waste-load-allocations-simplified-analytical-method-determining-npdes-effluent-limitations-potws-discharging-low-flow-streams>.

Water Environment Federation, *Manual of Practice 11 – Operation of Municipal Wastewater Treatment Plants*, Volumes I, II and III, 2007. <https://www.wefnet.org/ewef/images/MOP11/Frontmatter.pdf>.

Water Environment Federation, *Manual of Practice 8 – Design of Municipal Wastewater Treatment Plants*, Volumes I, II and III, 2010. <https://www.wefnet.org/ewef/images/mop8/mop8_frontmatter.pdf>.

Water Environment Federation, *Manual of Practice FD-12 - Alternative Sewer Systems*, 1986.

Water Environment Federation, *Manual of Practice No.8 - Design of Municipal Wastewater Treatment Plants*, Volumes I and II, 2010, <https://www.wefnet.org/ewef/images/mop8/mop8_frontmatter.pdf>.

Water Environment Federation, *Wastewater Treatment Plant Design - Manual of Practice*, 1975.

Water Environment Federation. *Standard Methods for the Examination of Water and Wastewater*, 23rd Edition. American Public Health Association, American Water Works Association, and Water Environment Federation, 2017. <https://connect.wef.org/s/store#/store/browse/detail/a2n1S000000KVk1>.

Water Supply Committee of the Great Lakes - Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers. "Recommended Standards for Water Works." *Policies for the Review and Approval of Plans and Specifications for Public Water Supplies.* Albany: Health Research Inc., Health Education Services Division, 2012. <https://www.health.state.mn.us/communities/environment/water/docs/tenstates/waterrev2012.pdf>

Yukon Territory Water Board, *Guidelines for Municipal Wastewater Discharges in the Yukon Territory*, March 1983.

Table of Contents

Chapter 6 Preliminary Treatment

This Chapter outlines physical unit operations to remove debris from wastewater that can cause operational problems with downstream processes or increase maintenance of downstream equipment. Preliminary treatment is typically considered the headworks of a plant and includes screening and grit removal.

6.1 Screening Devices

6.1.1 Bar Racks & Screens

6.1.1.1 Where Required

Coarse bar racks or screens should be provided as the first treatment stage for the protection of plant equipment against reduced operating efficiency, blockage, or physical damage. Screening of wastewater can be categorized according to screen opening size as follows:

- Trash racks and bypass screens: greater than 25 to 45 mm openings.
- Coarse screens: greater than 6.0 to 25 mm openings.
- Fine screens: greater than 1.0 to 6.0 mm openings.
- Micro-screens: 1.0 mm and smaller openings.

6.1.1.2 Selection Considerations

When considering which types of screening devices should be used, whether manually or mechanically screened, the following factors should be considered:

- Effect on downstream treatment and sludge disposal operations.
- Possible damage to comminutor or barminutor devices caused by stones or coarse grit particles.
- Head losses of the various alternative screening devices.
- Maintenance requirements.
- Screening disposal requirements, and quantities of screenings.
- Requirements for a standby unit.

Provision for the removal, drainage, washing, storage, and ultimate disposal of accumulated screenings should be provided when manually or mechanically cleaned screens are used (refer to Section 6.1.1.7).

6.1.1.3 Location

6.1.1.3.1 Outdoors

Screening devices installed outside should be protected from freezing. Where feasible screening devices should be located indoors.

6.1.1.3.2 Indoors

Screening devices installed in a building where other equipment or offices are located should be separated from the remainder of the building, provided with separate outside entrances, and provided with adequate means of ventilation. All electrical components should be suitable for the designated hazardous area classification.

6.1.1.3.3 Access
Screens located in pits more than 1.2 m deep should be provided with stairway access. Access ladders are acceptable for pits less than 1.2 m deep, in lieu of stairways.

6.1.1.3.4 Ventilation
Fresh air should be forced into enclosed screening device areas or into open pits more than 1.2 m deep. Dampers should not be used on exhaust or fresh air ducts and fine screens or other obstructions should be avoided to prevent clogging. Where continuous ventilation is required, at least 12 complete air changes per hour should be provided. Where continuous ventilation would cause excessive heat loss, intermittent ventilation of at least 30 complete air changes per hour should be provided when workmen enter the area.

Switches for operation of ventilation equipment should be marked and located conveniently. All intermittently operated ventilation equipment should be interconnected with the respective pit lighting system. The fan wheel should be fabricated from non-sparking material. Gas detectors should be provided in accordance with Section 5.8.

The minimum criteria for ventilation for protection against fire and explosion of wastewater treatment and pumping facilities should be in accordance with the latest edition of *NFPA 820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities* for the designated electrical classifications.

6.1.1.4 Design & Installation
6.1.1.4.1 Bar Spacing
Manually Cleaned Screens
Clear openings between bars should be from 25 mm to 45 mm. Design and installation should be such that they can be conveniently cleaned.

Mechanical Screens
Clear openings for mechanically cleaned screens may be as small as 15 mm.

Mechanical screens are recommended where the installation is not regularly supervised or where an increase in head results in plant bypass.

6.1.1.4.2 Velocities
At the design average rate of flow, the screen chamber should be designed to provide a velocity through the screen of approximately 0.3 m/s to prevent settling, and a maximum velocity during wet weather periods no greater than 0.75 m/s to prevent forcing material through the openings. The velocity should be calculated from a vertical projection of the screen openings on the cross-sectional area between the invert of the channel and the flow line.

6.1.1.4.3 Invert
The screen channel invert should be 75 to 150 mm below the invert of the incoming sewers. To prevent jetting action, the length and/or construction of the screen channel should be adequate to re-establish hydraulic flow pattern following the drop in elevation.

6.1.1.4.4 Slope
Manually cleaned screens, except those for emergency use, should be placed on a slope of 30 to 45° from horizontal. Mechanically cleaned screens should be placed on a slope of 45 to 90° from horizontal.

6.1.1.4.5 Channels
The channel preceding and following the screen should be shaped to eliminate stranding and settling of solids and should be designed to provide equal and uniform distribution of flow to the screens. Dual channels should be provided and equipped with the necessary gates to isolate flow from any screening unit. Provisions should also be made to facilitate dewatering each unit.

6.1.1.4.6 Flow Measurement
When flow measuring devices need to be in a screen channel, the effect of changes in backwater elevations, due to intermittent cleaning of screens, should be considered in locating of flow measurement equipment. The flow measurement devices should be selected based on reliability and accuracy.

6.1.1.5 Safety
6.1.1.5.1 Railings & Gratings
Manually cleaned screen channels should be protected by guard railings and deck gratings, with adequate provisions for removal or opening to facilitate raking.

Mechanically cleaned screen channels should be protected by guard railings and deck gratings. Consideration should also be given to temporary access arrangements to facilitate maintenance and repair.

6.1.1.5.2 Mechanical Devices
Mechanical screening equipment should have adequate removal enclosures to protect personnel against accidental contact with moving parts and to prevent dripping in multi-level installations.

A positive means of locking out each mechanical device and temporary access for use during maintenance should be provided.

6.1.1.5.3 Drainage
Floor design and drainage should be provided to prevent slippery areas.

6.1.1.5.4 Lighting
Suitable lighting should be provided in all work and access areas. Refer to Section 6.1.1.6.2.

6.1.1.6 Control Systems
6.1.1.6.1 Timing Devices
All mechanical units which are operated by timing devices should be provided with auxiliary controls which will set the cleaning mechanism in operation at pre-set high-water elevation. If the cleaning mechanism fails to lower the high water, a warning should be signaled.

6.1.1.6.2 Electrical Systems & Components
Electrical systems and components (e.g., motors, lights, cables, conduits, switchboxes, and control circuits) in enclosed or partially enclosed spaces where hazardous gases from elsewhere occasionally may be present

(including all space above raw or partially treated wastewater) should comply with the latest edition of *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations* and the regulations under the applicable provincial power standards. All electrical components must be consistent with the area classification as determined by the latest edition of *NFPA 820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities*.

6.1.1.6.3 Manual Override

Automatic controls should be supplemented by a manual override.

6.1.1.7 Screenings Removal & Disposal

A convenient and adequate means for removing screenings should be provided. Hoisting or lifting equipment may be necessary depending on the depth of pit and number of screenings or equipment to be lifted.

Facilities must be provided for handling, storage, and disposal of screenings in a manner acceptable to the Regulatory Authority. Separate grinding of screenings and return to the wastewater flow is unacceptable.

Manually cleaned screening facilities should include an accessible platform from which the operator may rake screenings easily and safely. Suitable drainage facilities should be provided for both the platform and storage area.

6.1.1.8 Auxiliary Screens

Where mechanically operated screening or comminuting devices are used, auxiliary manually cleaned screens should be provided. Where two or more mechanically cleaned screens are used, the design should provide for taking any unit out of service without sacrificing the capability to handle the peak design flow.

6.1.2 Fine Screens

6.1.2.1 General

Fine screens should not be considered equivalent to primary sedimentation but may be used in lieu of primary sedimentation providing that subsequent treatment units are designed based on anticipated screen performance. Where fine screens are used, additional provision for the removal of floatable oils and greases should be considered. Selection of screen capacity should consider flow restriction due to retained solids, gummy materials, frequency of cleaning and extent of cleaning.

6.1.2.2 Design

Tests should be conducted to determine BOD_5 and SS removal efficiencies at the design maximum day flow and design maximum day BOD_5 loadings. Pilot testing for an extended time is preferred.

A minimum of two fine screens should be provided, each unit being capable of independent operation. Capacity should be provided to treat design peak instantaneous flow with one unit out of service.

Fine screens should be preceded by a mechanically cleaned bar screen or other protective device. Fine screens should be protected from freezing and located to facilitate maintenance.

6.1.2.3 Electrical Fixtures & Control

Electrical fixtures and controls in screening areas where hazardous gases may accumulate should comply with the latest edition of *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations* and the regulations under the applicable provincial power standards. All electrical components must be consistent with the area classification as determined by the latest edition of *NFPA 820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities*.

6.1.2.4 Servicing

Hosing equipment (with hot and cold water) should be provided to facilitate cleaning. Provisions should be made for isolating or removing units from their location for servicing.

6.2 Comminutors/Grinders

6.2.1 General

Provisions for location should be in accordance with those for screening devices, refer to Section 6.1.1.3.

When required, comminutors or grinders should be used in plants that do not have primary sedimentation or fine screens and should be provided in cases where mechanically cleaned bar screens will not be used.

6.2.2 Design Considerations

6.2.2.1 Location

Comminutors or grinders should be located downstream of any grit removal equipment and be protected by a coarse screening device. Consideration for a different sequence may be given to suit individual cases.

6.2.2.2 Size

Comminutor or grinder capacity should be adequate to handle the design peak hourly flow.

6.2.2.3 Installation

A screened bypass channel should be provided. The use of the bypass channel should be automatic at depths of flows exceeding the design capacity of the comminutor or grinder.

Each comminutor or grinder that is not preceded by grit removal equipment should be protected by a 150 mm deep gravel trap.

Gates should be provided in accordance with Section 6.1.1.4.5.

6.2.2.4 Servicing

Provision should be made to facilitate servicing units in place and removing units from their location for servicing.

6.2.2.5 Electrical Controls & Motors

Electrical equipment in comminutor chambers where hazardous gases may accumulate should comply with the latest edition of *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations* and applicable provincial power standards.

Motors should be protected against accidental submergence and the potential impacts of climate change on processes leading to submergence.

6.3 Grit Removal Facilities

Grit removal is required in advance of treatment units to prevent the undue wear of machinery and the unwanted accumulation of solids in channels, settling tanks and digesters.

Grit removal facilities should be provided for all WWTPs and are required for plants receiving wastewater from combined sewers or from sewer systems receiving substantial amounts of grit. If a plant, serving a separate sewer system, is designed without grit facilities, the design should include provisions for future installation. Consideration should be given to possible damaging effects on pumps, comminutors and other preceding equipment, and the need for additional storage capacity in treatment units where grit is likely to accumulate.

The quantify of grit removed is variable and dependent on the wastewater flow, characteristics of the service area, type of collection system, and any upstream treatment such as screening. For a separated sewer system, the typical grit collection varies from 4 to 37 mL/m^3 of treated wastewater and from 4 to 180 mL/m^3 for CSSs.

6.3.1 Location

Grit removal facilities should be located ahead of pumps and comminuting devices. Coarse bar racks should be placed ahead of grit removal facilities.

6.3.2 Accessibility

Consideration should be given in the design of grit chambers to provide safe access to the chamber and, where mechanical equipment is involved, to all functioning parts.

6.3.3 Ventilation

Where grit removal units are installed indoor, uncontaminated air should be introduced continuously at a rate of 12 air changes per hour for a Class 1 Division 1 electrical rating or a less than 12 air changes per hour for Class 1 Division 1. The space would still be required to be ventilated at a higher rate intermittently when occupied if there is continuously less than 12 air changes per hour. Refer to the latest edition of *NFPA 820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities*, where the standard requires certain ventilation practices, they are intended to minimize fire and explosion hazards; these ventilation standards should not be considered to apply to the protection of personnel from the toxic effects of exposure to gases present or the depletion of oxygen.

6.3.4 Electrical

Electrical equipment in grit removal areas where hazardous gases may accumulate should comply with the latest edition of *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations* and the regulations under the applicable provincial power standards.

6.3.5 Outside Facilities

Grit removal facilities located outside should be protected from freezing.

6.3.6 Design Factors

6.3.6.1 Inlet

Inlet turbulence should be minimized in channel type units.

6.3.6.2 Type & Number of Units

Plants treating wastewater from combined sewers should have at least two mechanically cleaned grit removal units, with provisions for bypassing. A single manually or mechanically cleaned grit removal chamber with bypass is acceptable for small WWTPs (less than 3.8 m^3/day (1 MGD)) serving separate sanitary sewer systems. Facilities for larger plants (greater than 1 MGD) serving separate sanitary sewer systems should have at least one mechanically cleaned unit with a bypass. Facilities other than channel-types are desirable if provided with adequate and flexible controls for agitation and/or air supply devices and with grit collection and removal equipment.

6.3.6.3 Grit Channels

6.3.6.3.1 Velocity

Channel-type chambers should be designed to provide controlled velocities as close as possible to 0.30 m/s for normal variation in flow.

6.3.6.3.2 Weirs

Flow control sections should be of the proportional or Sutro Weir type.

6.3.6.3.3 Channel Dimensions

The minimum channel width should be 380 mm. The minimum channel length should be that required to settle a 0.2 mm particle with a specific gravity of 2.65, plus a 50% allowance for inlet and outlet turbulence.

6.3.6.3.4 Grit Storage

With permanently positioned weirs, the weir crest should be kept 150 to 300 mm above the grit channel invert to provide for storage of settled grit (weir plates that are capable of vertical adjustment are preferred since they can be moved to prevent the sedimentation of organic solids following grit cleaning). Grit storage is also a function of the frequency of grit removal.

6.3.6.4 Detritus Tanks

Detritus tanks should be designed with sufficient surface area to remove a 0.2 mm, or smaller, particle with a specific gravity of 2.65 at the expected peak flowrate. Detritus tanks, since they are mechanically-cleaned and do not need dewatering for cleaning, do not require multiple units, unless economically justifiable.

Separation of the organics from the grit before, during, or after the removal of the settled contents of the tank can be accomplished in one of the following ways:

- The removed detritus can be washed in a grit washer with the organic laden wash water being returned to the head of the detritus tank.
- A classifying-type conveyor can be used to remove the grit and return the organics to the detritus tank.
- The removed detritus can be passed through a centrifugal-type separator.

6.3.6.5 Aerated Grit Tanks

Aerated grit tanks for the removal of 0.2 mm, or larger, particles with specific gravity of 2.65, should be designed in accordance with the following parameters and as outlined in Table 6.1.

6.3.6.5.1 Air Supply

Air supply should be via air diffusers (wide band diffusion header) positioned lengthwise along one wall of the tank, 600 to 900 mm above the tank bottom. Air supply should be variable. Higher air supply rates should be used with tanks of large cross-section (i.e., greater than 3.6 m deep).

6.3.6.5.2 Inlet Conditions

Inlet flow should be parallel to induce roll pattern in tank. There should be a smooth transition from inlet to circulation flow.

6.3.6.5.3 Baffling

A minimum of one transverse baffle near the outlet weir should be provided. Additional transverse baffles in long tanks and longitudinal baffles in wide tanks should be considered.

6.3.6.5.4 Outlet Conditions

The outlet weir should be oriented parallel to the direction of induced roll (i.e., at a right angle to the inlet).

6.3.6.5.5 Tank Dimensions

The lower limit of the above aeration rates are generally suitable for tanks up to 3.6 m deep and 4.3 m wide. Wider or deeper tanks require aeration rates in the upper end of the below range. Long, narrow aerated grit tanks are generally more efficient than short tanks and produce a cleaner grit. A length to width ratio is normally 1.5-to-1.0 to 3.0-to-1.0, but up to 5.0-to-1.0 is acceptable. Depth to width ratios of 1.0-to-1.5 to 1.0-to-2.0 are acceptable.

6.3.6.5.6 Velocity

The surface velocity in the direction of roll in tanks should be 0.45 to 0.6 m/s (tank floor velocities will be approximately 75% of above). The velocity across the floor of the tank should not be less than 0.3 m/s.

6.3.6.5.7 Tank Geometry

“Dead spaces” in aerated grit tanks are to be avoided. Tank geometry is critical with respect to the location of the air diffusion header, sloping tank bottom, grit hopper and fitting of the grit collector mechanism into the tank structure. Consultation with equipment suppliers is advisable.

6.3.6.5.8 Multiple Units

Multiple units are generally not required unless economically justifiable, or where the grit removal method requires bypassing of the tank (as with clam shell bucket).

Table 6.1: Typical Design Information for Aerated Grit Chambers

Item	Value	
	Range	Typical
Detention time at peak flowrate (min)	2 – 5	3
Dimensions:		
Depth (m)	2 – 5	
Length (m)	7.5 – 20	
Width (m)	2.5 – 7	
Width:depth ratio	1:1 – 5:1	1.5:1
Length:width ratio	3:1 – 5:1	4:1
Air supply (m^3/min m of length)	0.2 – 0.5	
Grit quantities ($m^3/10^3\ m^3$)	0.004 – 0.200	0.015

* *Wastewater Engineering: Treatment and Resource Recovery* by Metcalf & Eddy (2014).

6.3.6.6 Mechanical Grit Chambers
Specific design parameters for mechanical grit chambers will be evaluated on a case-by-case basis.

6.3.6.7 Grit Washing
The need for grit washing should be determined by the method of grit handling and final disposal.

6.3.6.8 Dewatering
Provision should be made for isolating and dewatering each unit. The design should provide for complete draining and cleaning by means of a sloped bottom equipped with a drain sump.

6.3.6.9 Water
An adequate supply of water under pressure should be provided for cleanup.

6.3.7 Grit Removal
Grit facilities located in deep pits should be provided with mechanical equipment for pumping or hoisting grit to ground level. Such pits should have a stairway, approved-type elevator or man-lift, adequate ventilation, and adequate lighting.

6.3.8 Grit Handling
Grit removal facilities located in deep pits should be provided with mechanical equipment for hoisting or transporting grit to ground level. Impervious, non-slip, working surfaces with adequate drainage should be provided for grit handling areas. Grit transporting facilities should be provided with protection against freezing and loss of material.

6.3.9 Grit Disposal
Disposal of grit in approved sanitary landfills or lagoons, as well as grit incineration should be considered acceptable disposal methods. Whatever method of disposal is employed, the full spectrum of environmental considerations must be embodied in the final design.

6.4 Pre-Aeration & Flocculation

6.4.1 General

Pre-aeration of raw wastewater, may be used to achieve one or more of the following objectives:

- Odour control.
- Grease separation and increased grit removal.
- Prevention of septicity.
- Grit separation.
- Flocculation of solids.
- Maintenance of DO in primary treatment tanks at low flows.
- Increased removals of BOD and SS in primary units.
- Minimizes solids deposits on side walls and bottom of wet wells.

Flocculation of wastewater with or without coagulating aids, is worthy of consideration when it is desired to reduce the strength of wastewater prior to subsequent treatment. Also, flocculation may be beneficial in pre-treating wastewater containing certain industrial wastes.

6.4.2 Arrangement

The units should be designed so that removal from service will not interfere with normal operation of the remainder of the plant.

6.4.3 Pre-Aeration

6.4.3.1 Air Flow Measurements

Figure 6.1 represents air flow requirements for different periods of pre-aeration.

Pre-aeration periods should be 10 to 15 minutes if odour control and prevention of septicity are the prime objectives.

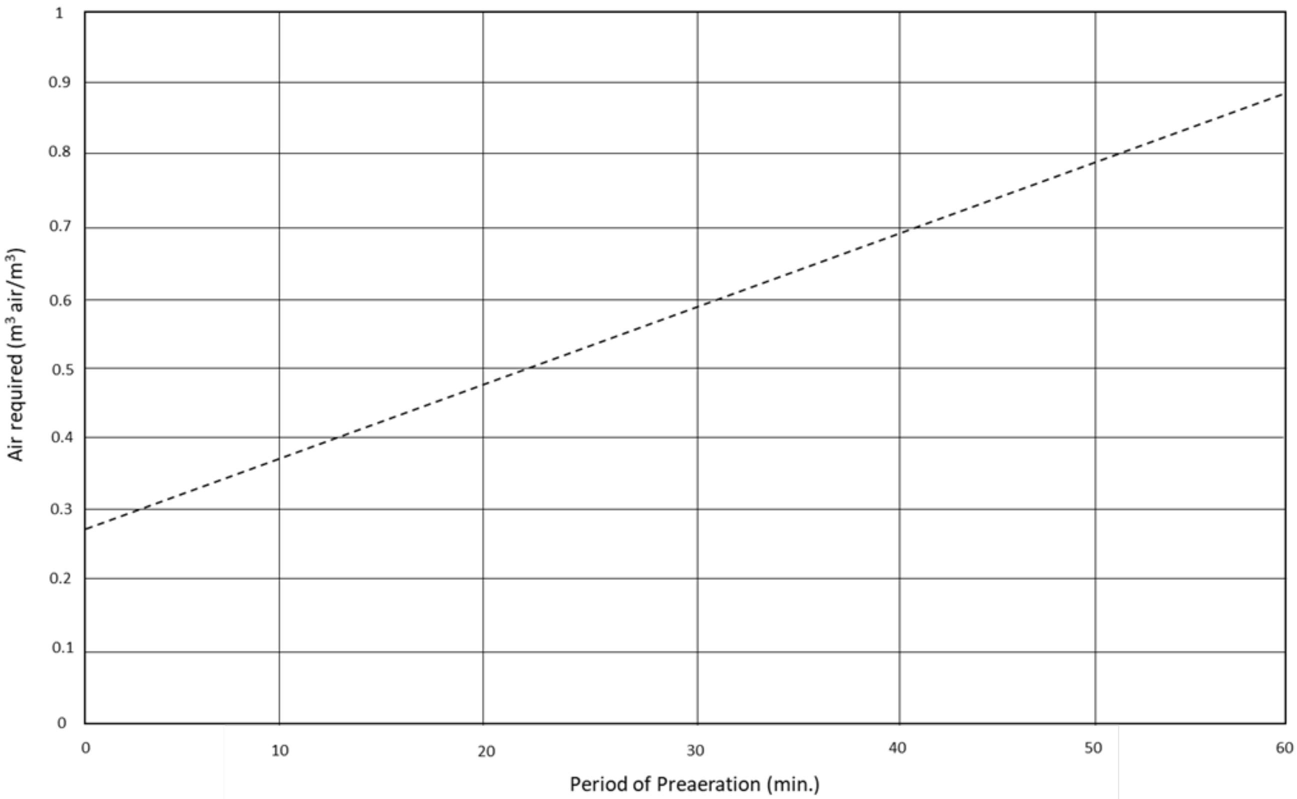


Figure 6.1: Air Flow Required for Different Periods of Pre-Aeration

6.4.4 Flocculation

6.4.4.1 Detention Period

When air or mechanical agitation is used in conjunction with chemicals to coagulate or flocculate the wastewater, the detention period should be approximately 30 minutes at the design flow, however, if polymers are used this may be varied.

6.4.4.2 Stirring Devices

6.4.4.2.1 Paddles

Paddles should have a peripheral speed of 0.50 to 0.75 m/s to prevent deposition of solids.

6.4.4.2.2 Aerators

Any of the types of equipment used for aerating activated sludge may be utilized. It should be possible to control agitation, to obtain good mixing and maintain self-cleaning velocities across the tank floor.

6.4.4.3 Details

Inlet and outlet devices should be designed to ensure proper distribution and to prevent short-circuiting. Convenient means should be provided for removing grit.

6.4.4.4 Rapid Mix

At plants where there are two or more flocculation basins utilizing chemicals, provision should be made for a rapid mix of the wastewater with the chemical so that the wastewater passing to the flocculation basins will be of uniform composition. The detention period provided in the rapid mixing chamber should be very short, 0.5 to 3 minutes.

6.5 Flow Equalization

6.5.1 General

Flow equalization can reduce the dry-weather variations in organic and hydraulic loadings at any WWTP. Flow equalization should be provided where large diurnal variations are expected and should include considerations for the impacts of climate change such as potential increases in the frequency and intensity of precipitation events contributing to I&I and leading to variations of wastewater flows.

6.5.2 Location

Equalization basins should be located downstream of pre-treatment facilities such as bar screens, comminutors and grit chambers.

6.5.3 Type

Flow equalization can be provided by using separate basins or online treatment units, such as aeration tanks. Equalization basins may be designed as either in-line or side-line units. Unused treatment units, such as sedimentation or aeration tanks, may be utilized as equalization basins during the early period of design life.

6.5.4 Size

Equalization basin capacity should be sufficient to effectively reduce expected flow and load variations to the extent deemed to be economically advantageous. With a diurnal flow pattern, the volume required to achieve the desired degree of equalization can be determined from a cumulative flow plot, or mass diagram, over a representative 24-hour period. To obtain the volume required to equalize the 24-hour flow:

- Draw a line between the points representing the accumulated volume at the beginning and end of the 24-hour period. The slope of this line represents the average rate of flow.
- Draw parallel lines to the first line through the points on the curve farthest from the first line.
- Draw a vertical line between the lines drawn in No. 2. The length of this line represents the minimum required volume.

6.5.5 Operation

6.5.5.1 Mixing

Where applicable, aeration or mechanical equipment should be provided to maintain adequate mixing. Corner fillets and hopper bottoms with draw-offs should be provided to alleviate the accumulation of sludge and grit.

6.5.5.2 Aeration

Where applicable, aeration equipment should be sufficient to always maintain a minimum of 1.0 mg/L of DO in the mixed basin contents. Air supply rates should be a minimum of 0.15 L/s/m^3 storage capacity. The air supply should be isolated from other treatment plant aeration requirements to facilitate process aeration control, although process air supply equipment may be utilized as a source of standby aeration.

6.5.5.3 Controls

Inlets and outlets for all basin compartments should be suitably equipped with accessible external valves, stop plates, weirs, or other devices to permit flow control and the removal of an individual unit from service. Facilities should also be provided to measure and indicate liquid levels and flowrates.

6.5.6 Electrical

All electrical work in housed equalization basins should comply with the latest editions of *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations* and *NFPA 820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities*, as well as the regulations under applicable provincial power standards.

6.5.7 Access

Suitable access should be provided to facilitate cleaning and the maintenance of equipment.

6.6 References

Ontario Ministry of the Environment. *Design Guidelines for Sewage Works*. Government of Ontario, 2019. Retrieved from <https://www.ontario.ca/document/design-guidelines-sewage-works-0>.

Tchobanoglous, George, Burton, Franklin L., Stensel, H. David, and Metcalf & Eddy. *Wastewater Engineering: Treatment and Reuse*. 4th Ed. / Revised by George Tchobanoglous, Franklin L. Burton, H. David Stensel. ed. McGraw-Hill Series in Civil and Environmental Engineering. Boston; Montreal: McGraw-Hill, 2003.

Wastewater Committee of the Great Lakes - Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers. *Recommended Standards for Wastewater Facilities*. Health Research, Inc., Health Education Services Division, 2014. <https://www.health.state.mn.us/communities/environment/water/docs/tenstates/tenstatestan2014.pdf>.

Table of Contents

Chapter 7 Clarification

7.1 Sedimentation Tanks

7.1.1 General Design Requirements

The need for and the design of primary sedimentation tanks will be influenced by various factors, including the following:

- The characteristics of the raw wastewater.
- The type of sludge digestion systems, either available or proposed (aerobic digestion should not be used with raw primary sludge).
- The presence, or absence, of secondary treatment following primary treatment.
- The need for handling of WAS in the primary settling tanks.
- The need for, or possible economic benefits through phosphorus removal in the primary settling tank(s).

7.1.1.1 Number of Units

Multiple units capable of independent operation are desirable and should be provided in all plants where design ADFs exceed 380 m^3/day. Plants not having multiple units should include other provisions to assure continuity of treatment.

7.1.1.2 Arrangement of Units

Settling tanks should be arranged in accordance with Section 5.4.5.

7.1.1.3 Interaction with Other Processes

- Pumping directly to any clarifier is prohibited unless special provision is included in the design of pump controls. Attention should be focused so that pumps deliver smooth flow transmissions at all times, with a minimal energy gradient.
- For activated sludge plants employing high energy aeration, provisions should be made for floc to be reformed before settling.
- For primary clarifiers, tanks and equipment must be sized to not only accommodate raw waste solids but also those solids introduced by thickener overflows, anaerobic digester overflow and sometimes WAS.

7.1.1.4 Flow Distribution & Control

Effective flow measurement devices and control appurtenances (e.g., valves, gates, and splitter boxes) should be provided to permit proper proportion of flow to each unit. Parallel basins should be of the same size, otherwise flow should be distributed in proportion to surface area.

7.1.1.5 Tank Configuration & Proportions

Consideration should be given to the probable flow pattern in the selection of tank size and shape, and inlet and outlet type and location. Generally rectangular clarifiers are designed with length-to-width ratios of at least 4-to-1, and width to depth ratios of 1-to-1 and 2.25-to-1.

7.1.1.6 Site Constraints

The selection of feasible clarifier alternatives should include the following site considerations:

- Wind direction.
- Proximity to residents.
- Soil conditions.
- Groundwater conditions, including considerations for the impacts of climate change (e.g., sea level rise impacts on the elevation and extent of the groundwater table).
- Available space.

7.1.1.7 Size Limitations

Rectangular clarifiers should have a maximum length of 90 m. Circular clarifiers should have a maximum diameter of 60 m. The minimum length of flow from inlet to outlet should be 3.0 m unless special provisions are made to prevent short circuiting. The vertical side water depth should be designed to provide an adequate separation zone between the sludge blanket and the overflow weirs. Generally, primary clarifiers have a side water depth between 3.0 to 4.6 m.

7.1.1.8 Inlet Structures

Inlet structures should be designed to dissipate the inlet velocity, to distribute the flow equally both horizontally and vertically and to prevent short-circuiting. Channels should be designed to maintain a velocity of at least 0.3 m/s at 1/2 design average flow. Corner pockets and dead ends should be eliminated, and corner fillets or channelling used where necessary. Provisions should be made for elimination or removal of floating materials in inlet structures.

7.1.1.9 Outlet Arrangements

7.1.1.9.1 General

Overflow weirs should be adjustable for levelling, and sufficiently long to avoid high heads which result in updraft currents.

7.1.1.9.2 Location

Overflow weirs should be located to optimize actual hydraulic detention time and minimize short circuiting. Peripheral weirs should be placed at least 0.3 m from the wall.

7.1.1.9.3 Weir Troughs

Weir troughs should be designed to prevent submergence at maximum design flow and to maintain a velocity of at least 0.3 m/s at 1/2 design average flow.

7.1.1.10 Submerged Surfaces

The tops of troughs, beams and similar submerged construction elements should have a minimum slope of 1.4 vertical to 1.0 horizontal. The underside of such elements should have a slope of 1.0 to 1.0 to prevent the accumulation of scum and solids.

7.1.1.11 Unit Dewatering

Unit dewatering features should conform to the provisions outlined in Section 5.4.3.5. The bypass design should also provide for redistribution of the plant flow to the remaining units.

7.1.1.12 Freeboard
Walls of settling tanks should extend at least 150 mm above the surrounding ground surface and should provide not less than 300 mm freeboard. Additional freeboard or the use of wind screens is recommended where larger settling tanks are subject to high velocity wind currents that would cause tank surface waves and inhibit effective scum removal.

7.1.1.13 Clarifier Covers
Clarifiers may be required to be covered for winter operation. The structure should be constructed with adequate head room for easy access. The structure must include adequate lighting, ventilation, and heating. Humidity and condensation should be controlled inside the structure and may impact selection of materials of construction.

7.1.2 Types of Settling

7.1.2.1 Type I Settling (Discrete Settling)
Type I settling is assumed to occur in gravity grit chambers handling wastewater and in basins used for preliminary settling (silt removal) of surface waters. A determination of the settling velocity of the smallest particle to be 100% removed is fundamental to the design of Type I clarifiers. Considering each particle is assumed to settle independently and with a constant velocity, a mathematical development is possible, based on Newton's Law and Stokes' Law.

7.1.2.2 Type II Settling (Flocculent Settling)
Type II settling occurs when particles initially settle independently but flocculate as they proceed to the bottom of the tank. As a result of flocculation, the settling velocities of the aggregates formed change with time, and a strict mathematical solution is not possible. Laboratory testing is required to determine appropriate values for design parameters. Type II settling can occur during clarification following fixed-film processes, primary clarification of wastewater, and clarification of potable water treated with coagulants.

Type II settling can also occur above the sludge blanket in clarifiers following activated sludge treatment, however, design procedures based on Type III settling are normally used to design these units.

7.1.2.3 Type III Settling (Hindered on Zone Settling)
Type III settling occurs in clarifiers following activated sludge processes and gravity thickeners. While Type II processes may occur to a limited extent in such units, it is Type III that governs design. In suspensions undergoing hindered settling, the solids concentration is usually much higher than in discrete or flocculent processes. As a result, the contacting particles tend to settle as a zone or blanket and maintain the same position relative to each other.

7.1.2.4 Type IV Settling (Compression Settling)
In Type IV settling, particles have reached such a concentration that a structure is formed and further settling, can only occur by compression. This type of settling typically occurs in the lower layers of a deep sludge mass such as near the bottom of secondary clarifiers and sludge thickeners.

7.1.3 Design Criteria

Table 7.1 and Table 7.2 display typical design information for primary sedimentation tanks. Table 7.3 displays typical design information for secondary clarifiers for the activated-sludge process. Table 7.4 displays the ranges of overflow rates and BOD and TSS removals from high-rate clarification processes treating wet-weather flows.

Table 7.1: Typical Design Information for Primary Sedimentation Tanks

Item	Unit	Range	Typical
Primary sedimentation tanks followed by secondary treatment			
Detention time	hour	1.5 - 2.5	2.0
Overflow rate			
Average flow	$m^3/m^2 \cdot d$	30 - 50	40
Peak hourly flow	$m^3/m^2 \cdot d$	80 - 120	100
Weir loading	$m^3/m \cdot d$	125 - 500	250
Primary settling with waste activated-sludge return			
Detention time	hour	1.5 - 2.5	2.0
Overflow rate			
Average flow	$m^3/m^2 \cdot d$	24 - 32	28
Peak hourly flow	$m^3/m^2 \cdot d$	48 - 70	60
Weir loading	$m^3/m \cdot d$	125 - 500	250

Table 7.2: Typical Dimensional Data for Rectangular & Circular Sedimentation Tanks for Primary Treatment

Item	Unit	Range	Typical
Rectangular			
Depth	m	3.0 - 4.9	4.3
Length	m	15 - 90	24 - 40
Width*	m	3 - 24	4.9 - 9.8
Flight speed	m/minute	0.6 - 1.2	0.9
Circular			
Depth	m	3.0 - 4.9	4.3
Diameter	m	3 - 60	12 - 45
Bottom slope	mm/mm	1/16 - 1/6	1/12
Flight speed	r/minute	0.02 - 0.05	0.03

*If widths of rectangular mechanically cleaned tanks are greater than 6.0 m, multiple bays with individual cleaning equipment may be used, thus permitting tank widths up to 24 m or more.

Table 7.3: Typical Design Information for Secondary Clarifiers for the Activated Sludge Process

Type of Treatment	Overflow Rate ($m^3/m^2 \cdot d$)		Solids Loading ($kg/m^2 \cdot h$)		Depth (m)
	Average	Peak	Average	Peak	
Settling following air-activated sludge (excluding extended aeration)	16 - 28	36 - 56	4.0 - 6.0	10	4.0 - 5.5
Selectors, biological nutrient removal	24 - 32	40 - 64	5.0 - 8.0	10	4.0 - 5.5
Settling following extended aeration	8 - 16	24 - 32	1.0 - 5.0	8	4.0 - 5.5
Settling for phosphorus removal effluent concentration (mg/L)					
Total phosphorus = 2.0	24 - 32				4.0 - 5.5
Total phosphorus = 1.0*	16 - 24				
Total phosphorus = 0.2 to 0.5**	12 - 20				

*Occasional chemical addition required.

**Continuous chemical addition required for effluent polishing.

Notes:

1. Peak is a 2-hour sustained peak.
2. Weir loading rates are used commonly in the design of clarifiers, although they are less critical in clarifier design than hydraulic overflow rates. Weir loading rates used in large tanks should preferably not exceed 375 m^3/lin m·d of weir at maximum flow when located away from the upturn zone of the density current, or 250 m^3/lin m·d when located within the upturn zone. In small tanks, the weir loading rate should not exceed 125 m^3/lin m·d at average flow or 250 m^3/lin m·d at maximum low. The upflow velocity in the immediate vicinity of the weir should be limited to about 3.5 to 7.0 m/hour.

Table 7.4: Rates and Removals from High-Rate Clarification Processes Treating Wet-Weather Flows

Parameter/Process	Ballasted Flocculation	Plate & Tube Settlers	Dense Sludge
Overflow rates			
Low ($m^3/m^2 \cdot d$)	1,200 - 2,900	880	2,300
Medium ($m^3/m^2 \cdot d$)	1,800 - 3,500	1,200	2,900
High ($m^3/m^2 \cdot d$)	2,300 - 4,100	1,800	3,500
BOD removals (%)			
At low overflow rates	35 - 50	45 - 55	25 - 35
At medium overflow rates	40 - 60	35 - 40	40 - 50
At high overflow rates	30 - 60	35 - 40	50 - 60
TSS removals (%)			
At low overflow rates	70 - 90	60 - 70	80 - 90
At medium overflow rates	40 - 80	65 - 75	70 - 80
At high overflow rates	30 - 80	40 - 50	70 - 80

7.1.4 Sludge & Scrum Removal

7.1.4.1 Scum Removal

Effective scum collection and removal facilities, including baffling, should be provided for all settling tanks. Scum baffles are to be placed ahead of the outlet weirs and extend 300 mm below the water surface. The unusual characteristics of scum which may adversely affect pumping, piping, sludge handling and disposal, should be recognized in design. Provisions may be made for the discharge of scum with the sludge, however, other special provisions for disposal may be necessary.

7.1.4.2 Sludge Removal

7.1.4.2.1 Sludge Removal

Sludge collection and withdrawal facilities should be designed to assure rapid removal of the sludge and minimization of density currents. Suction withdrawal should be provided for activated sludge plants designed for reduction of the nitrogenous oxygen demand and is encouraged for those plants designed for carbonaceous oxygen demand reduction. Each settling tank should have its own sludge withdrawal lines to ensure adequate control of the sludge wasting rate for each tank.

7.1.4.2.2 Sludge Collection

Sludge collection mechanisms should remain in operation during sludge withdrawal. Mechanism speeds should be such as to avoid undue agitation while still producing desired collection results.

7.1.4.2.3 Sludge Hopper

The minimum slope of the side walls should be 1.7 vertical to 1.0 horizontal. Hopper wall surfaces should be made smooth with rounded corners to aid in sludge removal. Hopper bottoms should have a maximum dimension of 0.6 m. Extra depth sludge hoppers for sludge thickening are not acceptable. The hoppers are to be accessible for sounding and cleaning.

7.1.4.2.4 Cross-Collectors

Cross-collectors serving one or more settling tanks may be useful in place of multiple sludge hoppers.

7.1.4.2.5 Sludge Removal Piping

Each hopper should have an individually valved sludge withdrawal line at least 150 mm in diameter. The static head available for withdrawal of sludge should be 760 mm or greater, as necessary to maintain a 0.9 m/s velocity in the withdrawal line. Clearance between the end of the withdrawal line and the hopper walls should be sufficient to prevent “bridging” of the sludge. Adequate provisions should be made for rodding or back-flushing individual pipe runs. Piping should also be provided to return sludge for further processing.

7.1.4.2.6 Sludge Removal Control

Separate settling tank sludge lines may drain to a common sludge well. Sludge wells equipped with telescoping valves or other appropriate equipment should be provided for viewing, sampling, and controlling the rate of sludge withdrawal from each tank hopper. The use of easily maintained sight glass and sampling valves may be appropriate. A means of measuring the sludge removal rate from each hopper should be provided. Air lift type of sludge removal will not be approved for removal of primary sludge. Sludge pump motor control systems should include time clocks and valve activators for regulating the duration and sequencing of sludge removal.

7.2 Enhanced Primary Clarification

7.2.1 Chemical Enhancement

Chemical coagulation of raw wastewater before sedimentation promotes flocculation of finely divided solids into more readily settleable flocs, thereby increasing SS, BOD, and phosphorus removal efficiencies. Sedimentation with coagulation may remove 60 to 90% of the TSS, 40 to 70% of the BOD, 30 to 60% of the

Chemical Oxygen Demand (COD), 70 to 90% of the phosphorus, and 80 to 90% of the bacteria loadings. In comparison, sedimentation without coagulation may remove only 40 to 70% of the TSS, 25 to 40% of the BOD, 5 to 10% of the phosphorus loadings, and 50 to 60% of the bacteria loading. Chapter 10 of these Guidelines contains additional information on the selection and application of chemicals for phosphorus removal.

Advantages of coagulation include greater removal efficiencies, the ability to use higher overflow rates, and more consistent performance. Disadvantages of coagulation include an increased mass of primary sludge, production of solids that are often more difficult to thicken and dewater, and an increase in operational cost and operator attention. The designer of chemical coagulation facilities should consider the effect of enhanced primary sedimentation on downstream solids-processing facilities.

7.2.1.1 Chemical Coagulants

Chemical coagulants such as ferric chloride and alum (typically < 60 mg/L) provide cations that destabilize colloidal particles in wastewater while flocculent aids such as polymer (typically < 2.0 mg/L), recycled sludge, and microsand function to accelerate the growth of floc, enlarge the floc, improve floc shape, strengthen floc structure, and increase particle specific gravity. The use of chemicals allows a higher peak overflow rate during peak events while maintaining or increasing primary clarifier performance, thus minimizing the clarifier surface area that must be provided for peak flows.

Chemically enhanced primary treatment has evolved over time. Early applications typically consisted of simply adding ferric chloride, alum, or lime to a conventionally designed primary settling tank. Only a few plants use lime as a coagulant for primary treatment since lime addition produces more primary sludge because of the chemical solids than do metals salts and lime is more difficult to store, handle, and feed. Coagulant selection for enhanced sedimentation should be based on performance, reliability, and cost. Performance evaluation should use jar tests of the actual wastewater to determine dosages and effectiveness. Operating experience, cost, and other relevant information drawn from other plants should be considered during selection. Current practice uses smaller metal salt doses (20 to 40 mg/L) in combination with polymer addition (< 1.0 mg/L) and includes the use of rapid mix and flocculation before the settling tank. Use of iron salts can decrease the efficiency of downstream disinfection with UV light.

7.2.1.2 Rapid Mix

During rapid mix, the first step of the coagulation process, chemical coagulants are mixed with the raw wastewater. The coagulants destabilize the colloidal particles by reducing the forces (zeta potential) keeping the particles apart, which allows their agglomeration. The destabilization process occurs within seconds of coagulant addition. At the point of chemical addition, intense mixing will ensure uniform dispersion of the coagulant throughout the raw wastewater. The intensity and duration of mixing must be controlled, however, to avoid overmixing or undermixing. Overmixing may reduce the removal efficiency by breaking up existing wastewater solids and newly formed floc. Undermixing inadequately disperses the chemical, increases chemical use, and reduces the removal efficiency.

The velocity gradient (G) is a measure of mixing intensity. Velocity gradients of 300 s^{-1} are typically sufficient for rapid mix, but some designers have recommended velocity gradients as high as 1,000 s^{-1}. Mechanical mixers, in-line blenders, pumps, baffled compartments, baffled pipes, or air mixers can accomplish rapid mix. The mixing intensity of mechanical mixers and in-line blenders is independent of flowrate, but these mixers cost considerably more than other types and might become clogged or entangled with debris. Air mixing eliminates the problem of debris and can offer advantages for primary sedimentation, especially if aerated channels or grit chambers already exist. Pumps, Parshall flumes, flow distribution structures, baffled compartments, or baffled pipes methods often used for upgrading existing facilities offer a lower-cost but less-efficient alternative to separate mixers for new construction. Methods listed above are less efficient than separate mixers because, unlike separate mixing, the mix intensity depends on the flowrate.

7.2.1.3 Flocculation

During the flocculation step of the coagulation process, destabilized particles grow and agglomerate to form large, settleable flocs. Through gentle prolonged mixing, chemical bridging and/or physical enmeshment of particles occurs. Flocculation is slower and more dependent on time and agitation than is the rapid-mix step. Typical detention times for flocculation range between 20 and 30 minutes. Aerated and mechanical grit chambers, flow distribution structures, and influent wells are areas that promote flocculation upstream of primary sedimentation. Advantages and disadvantages of different configurations resemble those for rapid-mix facilities.

Like rapid mix, the velocity gradient (G) achieved with each configuration should be checked. Velocity gradients should be maintained from 50 to 80 s^{-1}. Polymers are sometimes added during the flocculation step to promote floc formation. Polymers should enter as dilute solution to ensure thorough dispersion of polymers throughout the wastewater. Polymers may provide a good floc with only turbulence and detention in the sedimentation tank inlet distribution.

7.2.1.4 Coagulant Addition

Supplementing conventional primary sedimentation with chemical coagulation requires minimal additional construction. The optimal point for coagulant addition is as far upstream as possible from primary sedimentation tanks. The optimum feed point for coagulant addition often varies from plant to plant. If possible, several different feed points should be considered for additional flexibility. Dispersing the coagulant throughout the wastewater is essential to minimize coagulant dosage and concrete and metal corrosion associated with coagulant addition. Flow-metering devices should be installed on chemical feed lines for dosage control.

7.2.2 Plate & Tube Settlers

Plate and tube settlers are utilized to increase the effective settling area within the clarifier or settling basin. They can be used with or without chemical enhancement but typically are utilized in advanced primary applications. These types of settlers operate on the principle that by increasing the area where particles can settle within the settling unit using inclined tubes or plates will result in reduced footprint units accomplishing equivalent overflow rates to conventional settling basins with a much greater water surface area.

7.2.2.1 Calculation of Settling Area

The settling area within a plate clarifier is equal to the horizontally projected area of the vertically inclined plates, therefore, a settling basin equipped with (n) plates of overall surface area (A) inclined at an angle (∅) from the horizontal will have an equivalent settling area which can be calculated utilizing the equation:

$$Total\ Settling\ Area = nA(cos\emptyset)$$

Overflow rates can then be calculated utilizing the total settling area rather than the water surface area of the unit. Similar principles can be utilized for the calculation of total surface area and surface overflow rates for tube settlers.

7.2.2.2 Configuration

Typical settling plates are 0.2 m to 0.6 m wide and 3.0 m long with 50 mm spacing between multiple plates. Plate settlers are designed to operate in the laminar flow regime. Plate spacing must be large enough to prevent scouring of settled solids by the upward flowing liquid, to transport solids in a downward direction to the sludge hoppers, and to avoid plugging between the plates. In some instances, plate vibrators or mechanical scrapers can be utilized to prevent plugging. Flash mixers and flocculation chambers may be required ahead of the plate clarifier (as with all clarifiers) to mix in chemicals to promote floc growth and enhance the clarification process. Care must be taken to transport flocculated feed to the settling unit at less than 0.3 m/s to prevent floc breakup.

7.2.3 Ballasted Floc Clarifiers

The ballasted flocculation and settling process is a precipitative process which utilizes micro-sand combined with polymer for improved floc attachment and thus improved settling. The process involves:

- Coagulation.
- Injection.
- Maturation.
- Sedimentation.

During the coagulation process, metal-salt coagulants (typically alum or ferric sulphate) are added and thoroughly mixed into solution. The water then enters the injection chamber where polymer addition is followed by micro-sand injection and subsequent flash mixing. The maturation process acts like a typical flocculation chamber, utilizing an optimum mixing energy for optimized floc agglomeration onto the micro-sand.

In the settling process, water enters the lower region of the basin and travels through lamella plates. Solids collection with tube settlers in the bottom of the settling chamber is followed by cyclonic separation of micro-sand and sludge.

The micro-sand exiting the hydrocyclone is then re-injected into the treatment process. The micro-sand used typically has a diameter of 50 to 100 µm. The typical detention times for coagulation, injection, and maturation are 1 to 2 minutes, 1 to 2 minutes, and 4 to 6 minutes, respectively. The detention time of the settling basin depends on the rise rate, which is typically between 50 to 100 $m^3/m^2{\cdot}d$.

7.2.4 Dense-Sludge Process

The dense system is a proprietary process and differs from ballasted flocculation in that recycled chemically conditioned solids are used to form microfloc particles with the incoming wastewater entering an air-mixing zone where grit separation occurs and coagulant (usually ferric sulphate) is injected. After mixing, the wastewater flows into the first stage of a 2-stage flocculation tank where polymer is added together with chemically conditioned recirculated solids. Recirculated solids accelerate the flocculation process and ensure the formation of dense, homogeneous floc particles. In the second stage of flocculation, grease and scum begin separating and are removed. Flow from the flocculation tank enters a pre-settling zone and then passes into a plate settler. Most of the suspended flocculated solids are separated directly in the pre-settling zone; the residual flocculated particles are removed in the settler. A portion of the settled solids in recirculated, and the remainder is sent to the solids processing and disposal system.

7.3 Dissolved Air Flotation

Dissolved air flotation refers to the process of solids-liquids separation caused by the introduction of fine gas (usually air) bubbles to the liquid phase. The bubbles attach to the solids, and the resultant buoyancy of the combined solids-gas matrix causes the matrix to rise to the surface of the liquid where it is collected by a skimming mechanism.

Flotation can be employed in both liquid clarification and solids concentration applications. Flotator liquid effluent (known as subnatant) quality is the primary performance factor in clarification applications. These applications include flotation of refinery, meat-packing, meat-rendering, and other “oily” wastewaters. Float-solids concentrations are the main performance criteria in solids concentration flotation applications. Concentration applications include the flotation of waste solids from biological, mining, and metallurgical processes.

7.3.1 Process Design Considerations & Criteria

The feed solids to a DAF clarifier are typically mixed with a pressurized recycle flow before tank entry. The recycle flow is typically DAF tank effluent, although providing water from another source as a backup is often advisable if poor DAF performance causes an effluent high in SS. The recycle flow is pumped to an air saturation tank where compressed air enters and dissolves. As the pressurized recycle containing dissolved air is admitted back into the DAF tank (its surface is at atmospheric pressure), the pressure release from the recycle forms the air bubbles for flotation. A typical bubble-size distribution contains bubbles diameters ranging from 10 to 100 µm. Solids and air particles float and form a blanket on the DAF tank surface while the clarified effluent flows under the tank baffle and over the effluent weir. In general, the blanket on top of the DAF tank will be 150 to 300 mm thick.

Chemical conditioning with polymers is frequently used to enhance DAF performance. Polymer use significantly increases applicable solids-loading rates and solids capture but less effectively increases float-solids concentrations. If a polymer is used, it generally is introduced at the point where the recycle flow and the solids feed are mixed. Introducing the polymer solution into the recycle just as the bubbles are being formed are mixed with the solids produces the best results. Good mixing to ensure chemical dispersion while minimizing shearing forces will provide the best solids-air bubble aggregates.

Numerous factors affect DAF process performance, including:

- Type and characteristics of feed solids.
- Hydraulic Loading Rate (HLR).
- Solids-loading rate.
- Air-to-solids ratio.
- Chemical conditioning.
- Operating policy.
- Float-solids concentration.
- Effluent clarity.

7.3.1.1 Types of Solids

A variety of solids can be effectively removed by flotation. Among these are conventional activated sludge, solids from extended aeration and aerobic digestion, pure-oxygen activated sludge, and dual biological (trickling filter plus activated-sludge) processes.

Effects of the DAF process factors listed in the previous section make it difficult to document the specific performance characteristics of each of these types of solids. In other words, the specific conditions at each plant (for example, types of process, SRT, and Sludge Volume Indices (SVIs) in the aeration basin) dictate DAF performance to a greater extent than can be compensated for by flotation equipment adjustments such as air-to-solids ratio.

7.3.1.2 Hydraulic Loading Rate

Hydraulic loading rate refers to the sum of the feed and recycle flowrates divided by the net available flotation area. Dissolved air flotation clarifiers typically are designed for HLRs of 60 to 120 m/d, assuming no use of conditioning chemicals. The additional turbulence in flotators when the hourly HLR exceeds 5.0 m/hour may hinder the establishment of a stable float blanket and reduce the attainable float-solids turbulence forces the flow regime away from plug flow and more toward mixed flow. The addition of a polymer flotation aid generally is required to maintain satisfactory performance at hourly HLRs greater than 5.0 m/hour.

7.3.1.3 Solids-Loading Rate

The solids-loading rate of a DAF clarifier is generally denoted in terms of weight of solids per effective flotation area. With the addition of polymer, the solids-loading rate to a DAF thickener generally can be increased 50 to 100%, with up to a 0.5 to 1% increase in the thickened-solids concentration.

Operational difficulties may arise when the solids-loading rate exceeds approximately 10 kg/m^2h. The difficulties generally are caused by coincidental operation of excessive HLRs and by float-removal difficulties. Even in those instances when the hydraulic-loading rate can be maintained at less than 120 m/d, operation at solids-loading rates more than 10 kg/m^2h can cause float-removal difficulties. The increased amount of float created at high solids-loading rates necessitates continuous skimming, often at high skimming speeds.

Increased skimming speed, however, can cause float blanket disturbance and increase the amount of solids in the subnatant to unacceptable levels. In these circumstances, the addition of polymer flotation aid to increase the rise rate of the solids and the rate of float-blanket consolidation can alleviate some of the operating difficulties. Although stressed conditions, such as mechanical breakdown, excessive solids wastage, or adverse solids characteristics, may make it necessary to periodically operate in this manner, the flotation system should not be designed on this basis.

7.3.1.4 Feed-Solids Concentration

Changes in feed-solids concentration indirectly affect flotation in connection with the resultant changes in operating conditions. If the fed flowrate, recycle flow, pressure, and skimmer operations remain constant, an increase in feed-solids concentration results in a decrease in the air-to-solids ratio. Changes in feed-solids concentration, also result in changes to the float-blanket inventory and depth. Adjustments to the float skimmer speed may be required when operating strategy includes maintenance of a specific float-blanket depth or range of depths.

7.3.1.5 Air-to-Solids Ratio

The air-to-solids ratio is perhaps the single most important factor affecting DAF performance. It refers to the weight ratio of air available for flotation to the solids to be floated in the feed stream. Reported ratios range from 0.01-to-1.0 to 0.4-to-1.0. Adequate flotation is achieved in most municipal wastewater clarification applications at ratios of 0.02-to-1.0 to 0.06-to-1.0. Pressurization system sizing depends on many variables, including design solids loading, pressurization system efficiency, system pressure, liquid temperature, and concentration of dissolved solids. Pressurization system efficiencies differ among manufacturers and system configurations and can range from as low as 50% to more than 90%. Detailed information is available regarding the design, specification, and testing of pressurization systems.

Considering the float from a DAF clarifier contains a considerable amount of entrained air, this pumping application requires positive-displacement or centrifugal pumps that do not air bind, and special consideration of suction conditions. Initial density of the skimmed solids is approximately 700 kg/m^3. After the solids are held for a few hours, the air escapes and the solids return to normal densities. Float-solids content increases with increasing air-to-solids ratios up to a point where further increases in air-to-solids ratios result in only a nominal or no increase in float solids. The typical air-to-solids ratio at which float solids are maximized varies from 2 to 4%.

7.3.1.6 Float-Blanket Depth

The float produced during the flotation process must be removed from the flotation tank. The float-removal system usually consists of a variable-speed float skimmer and a beach arrangement. The volume of float that must be removed during each skimmer pass depends on the solids-loading rate, the chemical dosage rate, and the consistency of the float material.

Float-removal system skimmers are designed and operated to maximize float drainage time by incrementally removing only the top (driest) portion of the float and preventing the float blanket from expanding to the point where float exits the system in the subnatant. The optimal float depth varies from installation to installation. A float depth of 0.8 to 1.5 cm is typically sufficient to maximize float-solids content.

7.3.1.7 Polymer Addition

Chemical conditioning can enhance the performance of a DAF unit. Conditioning agents can be used to improve clarification and/or increase the float-solids concentration attainable with the unit. The amount of conditioning agent required, the point of addition (in the feed stream or recycle stream), and the method for intermixing should be specifically determined for each installation. Bench-scale flotation tests or pilot-unit tests provide the most effective method of determining the optimal chemical conditioning scheme for a particular installation. Typical polymer doses range from 2 to 5 g dry polymer/kg dry feed solids.

The addition of polymer usually affects solids capture to a greater extent than float-solids content. The float-solids content generally is increased up to 0.5% by the addition of dry polymer at a dosage of 2 to 5 g/kg dry solids.

If the lower ranges of hydraulic and solids loadings are used, the addition of polymer flotation aid typically is unnecessary for well-designed and operated DAF clarifiers. Maintenance of proper design and operating conditions as described in the preceding sections results in stable operation and satisfactory performance in terms of solids capture and float-solids concentration.

Solids recovery without polymer addition generally will be much greater than 90% when the DAF unit is sized as previously discussed. High loadings or adverse solids conditions can reduce solids recovery to 75 to 90%. Polymer-aided recovery can exceed 95%.

Under normal operations, the solids recycled from the DAF unit will not be damaging to the treatment system but will have the effect of increasing the WAS to be processed. In cases where the solids or hydraulic loading already are excessive, the recycled solids pose an additional burden on the system. Polymers should be employed under these conditions to maximize solids capture from the DAF unit.

7.4 Protection & Service Facilities

7.4.1 Operator Protection

All clarification tankage should be equipped to enhance safety for operators. Such features should appropriately include machinery covers, lifelines, stairways, walkways, handrails, and slip-resistant surfaces.

7.4.2 Mechanical Maintenance Access

The design should provide for convenient and safe access to routine maintenance items such as gear boxes, scum removal mechanisms, baffles, weirs, inlet stilling baffle area and effluent channels.

7.4.3 Electrical Fixtures & Controls

Electrical fixtures and controls in enclosed settling basins should comply with the latest edition of *CSA C22.1 Canadian Electrical Code, Part 1: Safety Standard for Electrical Installations* and the applicable provincial power standards. The fixtures and controls should be located to provide convenient and safe access for O&M. Adequate area lighting should be provided.

7.5 References

Ontario Ministry of the Environment. Design Guidelines for Sewage Works. Government of Ontario, 2019. Retrieved from <https://www.ontario.ca/document/design-guidelines-sewage-works-0>.

Tchobanoglous, George, Burton, Franklin L., Stensel, H. David, and Metcalf & Eddy. *Wastewater Engineering: Treatment and Reuse*. 4th Ed. / Revised by George Tchobanoglous, Franklin L. Burton, H. David Stensel. ed. McGraw-Hill Series in Civil and Environmental Engineering. Boston; Montreal: McGraw-Hill, 2003.

Wastewater Committee of the Great Lakes - Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers. 2014. *Recommended Standards for Wastewater Facilities*. Health Research, Inc., Health Education Services Division. <https://www.health.state.mn.us/communities/environment/water/docs/tenstates/tenstatestan2014.pdf>.

Water Environment Federation. *Clarifier Design: WRF Manual of Practice No. FD – 8*. McGraw-Hill Professional, 2005.

Table of Contents

Chapter 8 Biological Treatment

8.1 Activated Sludge

8.1.1 General

8.1.1.1 Applicability

The activated sludge process and its various modifications may be used where wastewater is amenable to biological treatment. This process requires close attention and competent operating supervision, including routine laboratory control. These requirements should be considered when proposing this type of treatment.

8.1.1.2 Specific Process Selection

The activated sludge process and its several modifications may be employed to accomplish varied degrees of removal of SS and reduction of BOD_5. Choice of the process most applicable will be influenced by the proposed plant size, type of waste to be treated, degree, and consistency of treatment required, anticipated degree of O&M and capital costs. All designs should provide for flexibility in operation.

8.1.1.3 Aeration Equipment Selection

Evaluation of aeration equipment alternatives should include the following considerations:

- Costs (capital and O&M).
- Oxygen transfer efficiency.
- Mixing capabilities.
- Diffuser clogging problems.
- Air pre-treatment requirements.
- Total power requirements.
- Aerator tip speed of mechanical aerators used with activated sludge systems.
- Icing problems.
- Misting problems.
- Cooling effects on aeration tank contents.

The size of the aeration tank for any particular modification of the process should be determined by full scale experience, pilot plant studies, or rational calculations based mainly on Food-to-Microorganism Ratio (F/M) or Mixed Liquor Suspended Solids (MLSS) levels. Other factors, such as size of treatment plant, diurnal load variations, and degree of treatment required, should also be considered. In addition, temperature, pH, and reactor DO should be considered when designing for nitrification.

8.1.1.4 Energy Requirements

This process requires major energy usage to meet aeration demands. Energy costs in relation to critical water quality conditions must be carefully evaluated. Capability of energy usage phase-down while still maintaining process viability, both under normal and emergency energy availability conditions, must be included in the activated sludge design.

8.1.1.5 Winter Protection

Protection against freezing should be provided to ensure continuity of operation and performance. Construction of enclosures should include climate and climate change considerations (e.g., insulation from cold weather, locking/securing mechanisms for high winds, enclosure flanges, or building joints at a level to prevent water ingress).

8.1.1.6 Pre-Treatment

Where primary settling tanks are not used, effective removal or exclusion of grit, debris, excessive oil or grease, and comminution or screening of solids should be accomplished prior to the activated sludge process.

Where primary settling is used, provision should be made for discharging raw wastewater directly to the aeration tanks to facilitate plant start-up and operation during the initial stages of the plant's design life.

8.1.1.7 Waste Activated Sludge Concentration

In the absence of sludge thickeners, other effective means of waste sludge concentration should be provided.

8.1.2 Process Definitions

The following are brief descriptions of a number of modified activated sludge process. See Section 8.2 for the Sequencing Batch Reactor (SBR) process description.

8.1.2.1 Conventional Plug Flow

The plug flow activated sludge process is a biological mechanism capable of removing 85 to 95% BOD_5 from typical municipal wastewater. The flow pattern is plug-flow-type. The process is characterized by 20 to 45% sludge return. This is the original activated sludge process and was later modified to suit various applications, situations, and treatment requirements. One characteristic of the plug flow configuration is a very high organic loading on the MLSS in the initial part of the task. Plug flow configurations are often preferred when high effluent DOs are sought.

8.1.2.2 Complete Mix Activated Sludge

In a Complete Mix Activated Sludge (CMAS) process, the characteristics of the mixed liquor are similar throughout the aeration tank. That is, the influent waste is rapidly distributed throughout the tank and the operating characteristics measured in terms of solids, Oxygen Uptake Rate (OUR), MLSS, and soluble BOD_5 concentration are identical throughout the tank. Considering the entire tank contents are the same quality as the tank effluent, there is a very low level of food available at any time to a large mass of microorganisms. This is the major reason why the complete mix modification can handle surges in the organic loading without producing a change in effluent quality.

8.1.2.3 Step Feed

Step feed is a modification of the plug flow configuration in which the secondary influent is fed at two or more points along the length of the aeration tank. With this arrangement, oxygen uptake requirements are relatively even, resulting in better utilization of the oxygen supplied. Step feed configurations generally use diffused aeration equipment. Secondary influent flow is usually added in the first 50 to 75% of the aeration tank's length.

8.1.2.4 Contract Stabilization

Contact stabilization activated sludge is both a process and a specific tankage configuration. Contact stabilization encompasses a short-term contact tank, secondary clarifier, and a sludge stabilization tank with about six times the detention time used in the contact tank.

This unit operation was developed to take advantage of the fact that BOD_5 removal occurs in two stages. The first is the absorptive phase and the second is the stabilization of the absorbed organics.

Contact stabilization is best for smaller flows in which the Mean Cell Residence Time (MCRT) desired is quite long, therefore, aerating return sludge can reduce tank requirements by as much as 30 to 40% vs. that required in an extended aeration system.

8.1.2.5 Extended Aeration

The extended aeration process uses the same flow scheme as the complete mix or plug flow processes but retains the wastewater in the aeration tank for long periods of time. This process operates at a high MCRT (low F/M) resulting in a condition where there is not enough food in the system to support all the microorganisms present. The microorganisms, therefore, compete very actively for the remaining food and even use their own cell structure for food. This highly competitive situation results in a highly treated effluent with low sludge production, however, extended aeration plant effluents generally have significant concentrations of "pin floc" resulting in BOD_5 and SS removals of about 85%. Many extended aeration systems do not have primary clarifiers. Also, many are package plants used by small communities.

The main disadvantages of this system are the large oxygen requirements per unit of waste entering the plant and the large tank volume needed to hold the wastes for the extended period.

8.1.2.6 Oxidation Ditch

The oxidation ditch is a variation of the extended aeration process. The wastewater is forced around a circular or oval pathway by a mechanical aerator/pumping device at one or more points along the flow pathway. In the aeration tank, the mixed liquor velocity is maintained between 0.20 to 0.37 m/s in the channel to prevent solids from settling.

Oxidation ditches use mechanical brush disk aerators, surface aerators, or jet aerator devices to aerate and propel the liquid flow.

8.1.2.7 High-Rate Aeration

This is a type of short-term aeration process in which relatively high concentrations of MLSS are maintained, by utilizing high sludge recirculation rates (100 to 500%), and low Hydraulic Retention Times (HRTs). Depending on the excess sludge wasting procedure, 60 to 90% BOD_5 removal is achieved for normal domestic wastes. This process is usually (but not necessarily) accomplished in "combined-tank" units.

8.1.2.8 High Purity Oxygen

The most common high purity oxygen activated sludge process uses a covered and staged aeration tank configuration. The wastewater, return sludge, and oxygen feed gas enter the first stage of this system and flow concurrently through the tank. The tanks in this system are covered to retain the oxygen gas and permit a high degree of oxygen use. A prime advantage of the staged reactor configuration of the oxygenation system is the system's ability to match approximately the biological uptake rate with the available oxygen gas purity.

8.1.2.9 Countercurrent Aeration

Countercurrent aeration is a unique aeration basin configuration which involves using a circular aeration basin with a centre-pivoted, traveling bridge supporting air diffusers. Rotating aerators continually re-suspend MLSS while leaving a veil of fine bubbles providing the aeration. Another set of fixed bubble aerators can also be provided. Its rising bubbles are swept along with the rotating liquid current induced by traveling diffusers. The rotating velocity of the liquid causes bubbles from both sources to lead or trail away from their point of release. Clean water efficiencies are typically lower than those of fine-pore disc/dome grid systems and similar to tube-grid arrangements. Considering additional energy is required to drive the bridge, it is likely that standard aeration efficiencies will be lower than those for more conventional activated sludge systems.

8.1.3 Return Sludge Equipment

8.1.3.1 Return Activated Sludge Rate

The minimum permissible return sludge rate of withdrawal from the final settling tank is a function of the concentration of SS in the mixed liquor entering it, the sludge volume index of these solids and the length of time these solids are retained in the settling tank. Since undue retention of solids in the final settling tank may be deleterious to both the aeration and sedimentation phases of the activated sludge process, the rate of RAS expressed as a percentage of the average design flow of wastewater should generally be variable between the limits set forth in the final column of Table 8.1.

The rate of sludge return should be varied by means of variable speed motors, drives or timers (small plants) to pump sludge at the above rates.

8.1.3.2 Return Sludge Pumps

If motor driven return sludge pumps are used, the maximum return sludge capacity should be obtained with the largest pump out of service. A positive head should be provided on pump suctions. Pumps should have at least 75 mm suction and discharge openings.

If air lifts are used for returning sludge from each settling tank hopper, no standby unit will be required, provided the design of the air lifts are such as to facilitate their rapid and easy cleaning and provided other suitable standby measures are provided. Air lifts should be at least 75 mm in diameter.

8.1.3.3 Return Sludge Piping

Suction and discharge piping should be at least 100 mm in diameter and should be designed to maintain a velocity of not less than 0.6 m/s and not more than 2.0 m/s, when return sludge facilities are operating at normal return sludge rates.

Suitable devices for observing, sampling, and controlling RAS flow from each settling tank should be provided.

8.1.3.4 Waste Sludge Facilities

Waste sludge control facilities should be designed for the maximum sludge production of the process. Means for observing, measuring, sampling, and controlling WAS flow should be provided. Waste sludge may be discharged to the primary settling tank, concentration or thickening tank, sludge digestion tank, mechanical dewatering facilities or any practical combination of these units.

8.1.3.5 Froth Control Units

It is essential to include some means of controlling froth formation in all aeration tanks. A series of spray nozzles may be fixed on top of the aeration tank. Screened effluent or potable water may be sprayed through these nozzles (either continuously or on a time clock on-off cycle) to physically break up the foam. Provisions may be made to use antifoaming chemical agents in the inlet of the aeration tank or preferably into the spray water.

8.2 Sequencing Batch Reactor

The SBR is a fill-and-draw activated sludge treatment system. All SBR systems utilize five steps that occur sequentially within the same tank as follows:

1. Fill.
2. React (aeration).
3. Settle (clarification).
4. Decant.
5. Idle.

Process modifications can be made by varying the times associated with each step in order to achieve specific treatment objectives. When designing or evaluating SBR systems, care must be taken with the processes that are unique to the SBR. These include:

- Fill method.
- Hydraulic control systems.
- Aeration control systems.
- Method of decant.
- Sizing of disinfection equipment for decant flows.
- Sludge wasting methods.

One of the main strengths of the SBR process is the process flexibility that can be achieved, therefore, the above processes can be performed using a variety of methods. Designers of SBR systems must be prepared to supply sufficient detailed information at the request of Regulatory Authority.

8.2.1 Process Configurations

One classification of SBR systems distinguishes those that operate with Continuous Feed and Intermittent Discharge (CFID) from those that operate with Intermittent Feed and Intermittent Discharge (IFID).

8.2.2 Continuous Influent Systems

Continuous feed and intermittent discharge reactors receive influent wastewater during all phases of the treatment cycle. When there is more than one reactor, as is typically the case for municipal systems, the influent flow is split equally to the various reactors on a continuous basis. For two reactor systems, it is normal to have the reactor cycle operations displaced so that one SBR is aerating while the second SBR is in the settling and decant phases. This makes it possible to aerate both reactors with one blower continuously in operation, while also spreading the decant periods so that there is no overlap. The dry weather flow cycle time for most CFID systems is generally 3 to 4 hours. Each cycle typically devotes 50% of the cycle time to aeration, 25% to settling, and 25% to decant. Storm water flows are accommodated by reducing cycle time. Under extreme flow condition, the reactor may operate as a primary clarifier (no aeration phase) with the decanters set at Top Water Level (TWL).

With a CFID system, TWL occurs at the start of the decant phase. Considering CFID systems generally operate on the basis of pre-set time cycles, TWL varies for each cycle as a function of the influent flow for that particular cycle. The actual effluent flowrate during the discharge event depends on the number of reactors and the percentage of each cycle devoted to decant.

A key design consideration with CFID systems is to minimize short-circuiting between influent and effluent. Influent and effluent discharges are typically located at opposite ends of rectangular reactors, with length-to-width ratios of 2-to-1 to 4-to-1 being common. Installation of a pre-reaction chamber separated by a baffle wall from the main reaction chamber is also a standard feature of some systems.

8.2.3 Intermittent Influent Systems

Intermittent feed and intermittent discharge types of systems are sometimes referred to as the conventional, or “true”, SBR systems. The one common characteristic of all IFID systems is that the influent flow to the reactor is discontinued for some portion of each cycle.

In IFID systems each reactor operates with five discrete phases during a cycle. During the period of reactor fill, any combination of aeration, mixing, and quiescent filling may be practiced. Mixing independent of aeration can be accomplished by using jet aeration pumps or separate mixers. Some systems distribute the influent over a portion of the reactor bottom so that it will contact settled solids during unaerated and unmixed fill. The end of the fill cycle is controlled either by time (that is, fill for a pre-set length of time), or by volume (that is, fill until the water level rises a fixed amount), or by a combination of both (fill to a max level or until the max fill time has elapsed). Flow information from the WWTP influent flow measurement or from the rise rate in the reactor, determined by a series of floats, may be used to control the time allocated to aeration, mixing, or filling in accordance with previously programmed instructions.

At the end of the fill cycle, all influent flow to the first reactor is stopped, and flow is diverted to the second reactor. Continuous aeration occurs during the react phase for a predetermined time (typically 1 to 3 hours). Again, the time devoted to reaction in any given cycle may automatically be changed as a function of influent flowrate. At the completion of the reaction phase, aeration and any supplemental mixing is stopped, and the mixed liquor is allowed to settle under quiescent conditions (typically 30 to 60 minutes). Next, clarified effluent is decanted until the Bottom Water Level (BWL) is reached. The idle period represents that time between the end of decant and the time when influent flow is again redirected to a given reactor. During high-flow periods, the time in idle will typically be minimal.

The actual flowrate during discharge has the potential to be several times higher than the influent flowrate. Discharge flowrates are critical design parameters for the downstream hydraulic capacity of sewers (in the case of industrial treatment facilities) or processes such as disinfection or filtration. Decant rates should not cause disturbance of the settled sludge in the SBR.

Another variation of the IFID approach dispenses with a dedicated reaction phase and initiates the settling cycle at the end of aerated fill. Yet another IFID approach allows influent to enter the reactor at all times except for the decant phase so that normal system operation consists of the following phases:

1. Fill-aeration.
2. Fill-settling.
3. No fill-decant.
4. Fill-idle.

These systems also include an initial selector compartment that operates either at constant or variable volume and serves as a flow splitter in multiple-basin systems. Biomass is directed from the main aeration zone to the selector.

Sequencing batch reactor systems can also be designed for nitrification-denitrification and enhanced biological phosphorus removal. In these cases, the cycle times devoted to such processes as anaerobic fill, anoxic fill, mixed/unmixed fill, aerobic fill, and dedicated reaction depend on the treatment objectives. Mineral addition may also be practiced to achieve effluent objectives more stringent than typical secondary effluent requirements. Systems can also be configured to switch from IFID operation to CFID operation when necessary to accommodate storm water flows or to allow a basin to be removed from service while still treating the entire WWTP flow in a remaining basin. The one common factor behind all SBRs is that aeration, settling, and decant occur within the same reactor.

8.2.4 Sequencing Batch Reactor Equipment

8.2.4.1 Process Control

The PLC is the optimum tool for SBR control and all present-day vendors use this approach. Sequencing batch reactor manufacturers supply both the PLC and required software. Typically, programs are developed and modified by the SBR vendor using a desk-top computer and software supplied by the PLC vendor. Vendor-developed programs are proprietary and may not be modified by the Design Engineer or the WWTP operator. Depending on the proprietary software design and type of system, the operator may independently select such variables as solids waste rates, storm cycle times, and aeration, mixing, settling, and idle times. In addition, Human Machine Interface (HMI) programs are available to present operating data and trends to the operator, to allow the operator to make adjustments in set points, to respond to alarm annunciation, and to generate reports. Through high-speed internet or telephone/modem access, the operator is able to monitor and adjust the plant operations remotely.

Programmable logic controller hardware is of modular construction. Troubleshooting procedures are well defined, and replacement of a faulty module is not difficult. An internal battery, UPS, protects the software in the event of power failure. The software is backed up by a memory chip (EPROM) and can be easily reloaded if the battery fails. The PLC expertise required of the System Owner/operators is limited to maintenance and repair functions that are well within the capability of a competent electrician.

Unless influent and effluent flow equalization is provided, the aeration system should be designed to match the diurnal organic load variation.

8.2.4.2 Reactors

Reactor shapes include rectangular, oval, circular, sloped sidewall, and other unique approaches. Design TWLs and BWLs often allow decanting from 20 to 30% of the reactor contents per cycle.

8.2.4.3 Decanters

Some decanters are mechanically actuated surface skimmers that typically rest above the TWL. The decanter is attached to the discharge pipe by smaller pipes that both support and drain the decanter. The discharge pipe is coupled at each end through seals that allow it to rotate. A screw-type jack attached to a worm gear, sprocket, and chain to an electric motor rotates the decanter from above the TWL to BWL. The speed of rotation is adjustable.

Other decanters are floated on the reactor surface. These decanters may approximate a large-diameter plug valve, whereby the top portion acts as the valve seat (and provides flotation). The bottom is the plug that is connected to a hydraulic operator that moves it away from the seat to allow discharge, or back to the seat to stop discharge. Other floating decanters consist of a length of pipe suspended on floats, with the pipe having a number of orifices bored in the bottom. The number of orifices (and length of pipe) is flow dependent. Each orifice is blocked by a flapper or plugs to prevent solids entry during aeration. There are also decanter configurations that float an effluent discharge pump.

Decanters that draw the treated effluent from near the water surface throughout the decant phase are recommended, as these maximise the settling time.

Other decanters are typically fixed-position siphons located on the reactor wall. The bottom of the decanter (collection end of the siphon) is positioned at the BWL. Flow into the decanter is under a front lip (scum baffle), over an internal dam, and out through a valve. When the water level in the reactor falls below the front lip, air enters the decanter, breaking the siphon and stopping flow. The trapped air prevents mixed liquor from entering during the reaction and settling modes. At the end of settling, the trapped air is released through a solenoid valve and the siphon is started.

8.2.4.4 Solids Wasting

The wasting of both aerated and settled MLSS is practiced. The wasting systems frequently consist of a submersible pump with a single point for withdrawal. Gravity flow waste systems are also used. Another approach uses influent distribution piping for multiple point with withdrawal of the settled solids.

8.2.4.5 Aeration/Mixing Systems

A variety of aeration and mixing systems are in use with SBRs. These include jet aeration, fine- and course-bubble aeration, hypermixers, and turbine mechanical aeration. Some systems use a floating mixer to provide mixing independent of aeration. Other diffused aeration facilities do not have any mixing capability independent of aeration. Independent mixing is readily obtained with a jet aeration system.

Where applicable, blowers should be provided in multiple units, so arranged and in such capacities as to meet the maximum air demand in the toxic portions of the fill/react and react phases of the cycle with the single largest unit out of service.

8.3 Activated Sludge Design Parameters

To maintain high levels of treatment performance with the activated sludge process under a wide range of operating conditions, special attention must be given to process control. The principal approaches to process control are:

- Maintaining a target Solids Retention Time (SRT).
- Maintaining a target DO level in the aeration tank.
- Regulating the RAS flowrate.

Other approaches include control of the MLSS in the aeration basin by regulation of the WAS.

Typical design parameters for activated sludge process modifications are indicated in Table 8.1.

Table 8.1: Typical Design Parameters for Activated Sludge Process Modifications

Process Name	Type of Reactor	SRT (days)	F/M (kg BOD/kg MLVSS·d)	Volumetric Loading (kg BOD/m^3/day)	MLSS (mg/L)	Detention Time (hours)	RAS % of Influent[b]
High-rate aeration	Plug flow	0.5 - 2	1.5 - 2.0	1.2 - 2.4	200 - 1,000	1.5 - 3	100 - 150
Contact stabilization	Plug flow	5 - 10	0.2 - 0.6	1.0 - 1.3	1,000 - 3,000[c] 6,000 - 10,000[d]	0.5 - 1[c] 2 - 4[d]	50 - 150
High-purity oxygen	Plug flow	1 - 4	0.5 - 1.0	1.3 - 3.2	2,000 - 5,000	1 - 3	25 - 50
Conventional plug flow	Plug flow	3 - 15	0.2 - 0.4	0.3 - 0.7	1,000 - 3,000	4 - 8	25 - 75[e]
Step feed	Plug flow	3 - 15	0.2 - 0.4	0.7 - 1.0	1,500 - 4,000	3 - 5	25 - 75
Complete mix	CMAS	3 - 15	0.2 - 0.6	0.3 - 1.6	1,500 - 4,000	3 - 5	25 - 100[e]
Extended aeration	Plug flow	20 - 40	0.04 - 0.1	0.1 - 0.3	2,000 - 5,000	20 - 30	50 - 150
Oxidation ditch	Plug flow	15 - 30	0.04 - 0.1	0.1 - 0.3	3,000 - 5,000	15 - 30	75 - 150
SBR	Batch	10 - 30	0.04 - 0.1	0.1 - 0.3	2,000 - 5,000[f]	15 - 40	N/A
Counter-current aeration system	Plug flow	10 - 30	0.04 - 0.1	0.1 - 0.3	2,000 - 4,000	15 - 40	25 - 75[e]

a. Adapted from Water Environment Federation, Wastewater Treatment Plant Design, 2003.
b. Based on average flow.
c. Mixed liquor suspended solids and detention time in contact basin.
d. Mixed liquor suspended solids and detention time in stabilization pond.
e. For nitrification, rates may be increased by 25 to 50%.
f. Also used at intermediate SRTs.
g. MLVSS - Mixed Liquor Volatile Suspended Solids.

The size of the aeration tank for any particular adaptation of the process should be determined by full scale experience, pilot plant studies, or rational calculations based mainly on food to microorganism ratio and MLSS levels. Other factors, such as size of treatment plant, diurnal load variations, and degree of treatment required, should also be considered. In addition, temperature, pH, and reactor DO should be considered when designing for nitrification.

8.4 Aeration

8.4.1 Arrangement of Aeration Tanks

8.4.1.1 General Tank Configuration

Dimensions

The dimensions of each independent mixed liquor aeration tank or return sludge re-aeration tank should be such as to maintain effective mixing and utilization of air.

Aeration basin depth is an important consideration in the design of aeration systems because of the effect that depth has on the aeration efficiency and air pressure requirements of diffused aeration devices and mixing capabilities of mechanical aerators. A minimum aeration basin depth of 3.0 to 4.6 m is recommended for typical WWTPs. Oxidation ditches should have minimum depth of 1.6 m.

Short-Circuiting

For very small tanks or tanks with special configuration, the shape of the tank, the location of the influent and sludge return and the installation of aeration equipment should provide for positive control of short-circuiting through the tank.

Number of Units

Total aeration tank volume should be divided among two or more units, capable of independent operation, when the total aeration tank volume required exceeds 140 m^3.

8.4.1.1.1 Inlets & Outlets

Controls

Inlets and outlets for each aeration tank unit should be suitably equipped with valves, gates, stop plates, weirs, or other devices to permit controlling the flow to any unit and to maintain a reasonably constant liquid level. The hydraulic properties of the system should permit the design peak instantaneous hydraulic load to be carried with any single aeration tank unit out of service. The effluent weir for an oxidation ditch must be easily adjustable by mechanical means.

Conduits

Channels and pipes carrying liquids with solids in suspension should be designed to maintain self-cleansing velocities or should be agitated to keep such solids in suspension at all rates of flow within the design limits. Adequate provisions should be made to drain segments of channels which are not being used due to alternate flow patterns.

8.4.1.1.2 Measuring Devices

Devices should be installed for indicating flowrates of raw wastewater or primary effluent, return sludge and air to each tank unit. For plants designed for wastewater flows of 5,000 m^3/d or more, these devices should totalize and record, as well as indicate flows. Where the design provides for all return sludge to be mixed with the raw wastewater (or primary effluent) at one location, then the mixed liquor flowrate to each aeration unit should be measured.

8.4.1.1.3 Freeboard

All aeration tanks should have a freeboard of not less than 450 mm. Additional freeboard or windbreak may be necessary to protect against freezing or windblown spray. If a mechanical surface aerator is used, the freeboard should not be less than 900 mm.

8.4.1.2 Aeration Equipment

8.4.1.2.1 General

Oxygen requirements generally depend on maximum BOD_5 loading, degree of treatment and level of SS concentration to be maintained in the aeration tank mixed liquor. Aeration equipment should be capable of maintaining a minimum of 2.0 mg/L of DO in the mixed liquor at all times and providing thorough mixing of the mixed liquor. In the absence of experimentally determined values, the design oxygen requirements for all activated sludge processes should be 1.1 kg O_2/kg peak BOD_5 applied to the aeration tanks with the exception of the extended aeration process, for which the value should be 1.5 kg O_2/kg peak BOD_5. In the case of nitrification, the oxygen requirement for oxidizing ammonia must be added to the above requirement for carbonaceous BOD_5 removal. The Nitrogen Oxygen Demand (NOD) should be taken as 4.6 times the diurnal peak TKN content of the influent. In addition, the oxygen demands due to recycle flows (heat treatment supernatant, vacuum filtrate, elutriates, etc.) must be considered due to the high concentrations of BOD_5 and TKN associated with such flows.

Careful consideration should be given to maximizing oxygen utilization per unit power input. Unless flow equalization is provided, the aeration system should be designed to match diurnal organic load variation while economizing on power input.

8.4.1.2.2 Variable Oxygenation Capacity

Consideration should be given to reducing power requirements of aeration systems by varying oxygenation capacity to match oxygen demands within the system. Such a system would utilize automatic DO probes in each aeration basin to measure DO levels.

8.4.1.2.3 Mixing Requirements

The aeration system which is selected must not only satisfy the oxygen requirements of the mixed liquor but must also provide sufficient mixing to ensure that the mixed liquor remains in suspension. The power levels necessary to achieve uniform DO and MLSS concentrations are shown in Table 8.2.

Table 8.2: Aeration Mixing Requirements

Aeration System	For Uniform DO Levels	For Uniform MLSS Levels
Mechanical	1.6 to 2.5 W/m^3	16 to 25 W/m^3
Diffused (coarse bubble, spiral roll, etc.)	-	0.33 $L/m^3/s$
Diffused (fine bubble domes, full floor coverage, etc.)	-	0.6 $L/m^2/s$

Notes:

- Mixing requirements vary with basin geometry, MLSS concentrations, placement of aeration devices, pumping efficiency of aerators, etc. Wherever possible, refer to full-scale testing results for the aerator being considered.
- $L/m^3/s$ refers to volume of air per second per volume of aeration tank.
- $L/m^2/s$ refers to volume of air per second per horizontal cross-sectional area of aeration tank.

8.4.1.2.4 Back-up Requirements

Aeration systems will require facilities to permit continuous operation, or minimal disruption, in the event of equipment failure. The following factors should be considered when designing the back-up requirements for aeration systems:

- Effect on the aeration capacity if a piece of equipment breaks down, or requires maintenance (for instance, the breakdown of one of two blowers will have a greater effect on capacity than the breakdown of one of four mechanical aerators).
- Time required to perform the necessary repair and maintenance operations.
- The general availability of spare parts and the time required to obtain delivery and installation.
- Means other than duplicate equipment to provide the necessary capacity in the event of a breakdown (for instance, using over-sized mechanical aerators with adjustable weirs to control power draw and oxygenation capacity, or using two speed mechanical aerators, etc.).

Generally, considerations such as the above will mean that diffused aeration systems will require a standby blower but mechanical aeration systems may not require standby units, depending upon the number of duty units, availability of replacement parts, etc.

8.4.1.2.5 Oxygen Transfer & Oxygen Transfer Efficiency

Aeration equipment must be designed to carry out its functions under conditions much different than those under which it may be tested by the equipment supplier. The bulk of oxygen transfer tests are conducted under conditions commonly referred to as standard or are corrected to standard conditions.

The designer, therefore, must test the unit under standard conditions and project its efficiency to the mixed liquor, or conduct the test in the mixed liquor. In either case, there are intricacies (related to aerator testing) involved in making this conversion from one condition to another. It is good practice to work with the suppliers when selecting aerators to discuss and agree on, at the time of design, the planned test procedure and interpretation of results.

It is most common to express the oxygenation rate of a particular activated sludge aeration device either as Standard Oxygen Rate (SOR) or Actual Oxygen Rate (AOR), both in kg of oxygen transferred per hour (kgO_2/hour). Either value is considered to be determinable, given the other and its transfer environment. Methods used to calculate oxygen transfer for conditions other than standard, or to correct to standard conditions the results obtained by mixed liquor testing are shown in Table 8.3.

Table 8.3: Methods to Calculate Oxygen Transfer

Mechanical Surface Aerators			Diffused Air and Submerged Turbine Aerators
Equation 1: $AOR = \alpha(SOR)\ (ß(C_{sw}-C_0)/C_s)\ \Theta^{(T-20)}$			Equation 2: $AOR = \alpha(SOR)\ (ß(C_{SC*}-C_0)/C_s)\ \Theta^{(T-20)}$
α	=	Relative rate of oxygen transfer as compared to clean water, dimensionless equal to 1.0 under standard conditions. Mixed liquor range is 0.6 (near basin influent) to 0.94, with the higher value representative of well-treated wastes. Values generally range from 0.8 to 0.94. Initial stages of plug flow systems may have very low α; industrial wastes may reduce α.	
ß	=	Relative oxygen saturation value as compared to clean water, dimensionless (equal to 1.0 under standard conditions. Mixed liquor range to 0.9 to 0.97, with upper level seen in well-treated wastes).	
Θ	=	Temperature correction constant (1.024).	
C_s	=	Oxygen saturation value of clean water at standard conditions. C_s = 9.17 mg/L	
C_{sw}	=	Saturation value of clean water at the surface, at site conditions of temperature (T) and actual barometric pressure (P_a).	
$C_{sc}*$	=	Corrected Cs value for water depth (D) and oxygen content of gaseous phase (mg/L).	
C_0	=	Initial (or steady state) DO level mg/L (equal to 0 mg/L under standard conditions. Mixed liquor values range from 1.5 to 2.0 mg/L at average oxygen uptake conditions).	
T	=	Temperature of bulk liquid, °C (equal to 20°C under standard conditions). Mixed liquor values range from 5 to 30°C; highest operating temperature is most conservative in terms of design (C_s and $\Theta^{(20-T)}$ to self-correct).	

In the absence of experimentally determined α and ß factors, wastewater transfer efficiency should be assumed to be 50% of clean water efficiency for plants treating primarily (90% or greater) domestic wastewater. Treatment plants where the waste contains higher percentages of industrial wastes should use a correspondingly lower percentage of clean water efficiency and should have calculations submitted to justify such a percentage.

Oxygen transfer efficiencies are generally represented in terms of kg of O_2 transferred per MJ. Manufacturers of aeration equipment will generally designate the specific equipment O_2 transfer rate as Ns (the standard transfer efficiency or rated capacity). The rated capacity can be expressed as:

Equation 3:

$$Ns = \frac{SOR}{\begin{array}{c}Total\ Power\ Consumed \\ (by\ mixer/aerator, pump, blower)\end{array}}$$

The designer must, therefore, use Equation 3 to determine the AOR as described in Equation 1 or Equation 2 (in Table 8.3).

8.4.1.2.6 Characteristics of Aeration Equipment

Table 8.4 outlines various characteristics of some typical aeration equipment.

8.4.1.2.7 Diffused Air Systems

Typical air requirements for all activated sludge processes except extended aeration (assuming equipment capable of transmitting to the mixed liquor the amount of oxygen required in Section 8.4.1.2.1) is 100 m^3 per kg of BOD_5 peak aeration tank loading. For the extended aeration process the value is 125 m^3/kg BOD_{5-} peak.

Air requirements for diffused air systems should be augmented as required considering the following items:

- To the air requirements calculated should be added air required for channels, pumps, aerobic digesters, or other air-use demand.
- The specified capacity of blowers or air compressors, particularly centrifugal blowers, should consider that the air intake temperature may reach 40°C or higher and the pressure may be less than normal. The specified capacity of the motor drive should also consider that the intake air may be -30°C or less and may require oversizing of the motor or a means of reducing the rate of air delivery to prevent overheating or damage to the motor.
- The blowers should be provided in multiple units, so arranged and in such capacities as to meet the maximum air demand with the single largest unit out of service. The design should also provide for varying the volume of air delivered in proportion to the load demand of the plant. Aeration equipment should be easily adjustable in increments and should maintain solids suspension within these limits.
- Diffuser systems should be capable of providing for the diurnal peak oxygen demand or 200% of the design average day oxygen demand, whichever is larger. The air diffusion piping and diffuser system should be capable of delivering normal air requirements with minimal friction losses.

Table 8.4: Characteristics of Aeration Equipment

Equipment Type	Equipment Characteristics	Process Where Used	Advantages	Disadvantages	Reported Transfer Efficiency* (kg/MJ) for Standard Conditions, 0 DO, 20°C, 101 kPa, & Clean Water
Diffused air: A) Bubbler porous diffusers	• Produce fine-to-medium bubbles. • Made of ceramic domes, plates, tubes, plastic-cloth tube, or bag.	• High rate. • Conventional extended. • Step. • Modified. • Contact-stabilization. • Activated sludge process.	• Good mixing. • Maintains liquid temperature. • Varying air flow provides good operational flexibility.	• High initial and maintenance costs. • Air filters needed. • Spiral configuration limits tank geometry.	0.31 - 0.42
Nonporous diffusers	• Made in bubble cap, nozzle, valve, orifice, or shear types. • They produce coarse or large bubbles. • Some made of plastic with check valve design.	• Same as for porous diffusers.	• Non-clogging. • Maintains liquid temperature. • Low maintenance cost.	• High initial cost. • Low oxygen transfer efficiency. • High power cost. • Fouling may occur.	0.20 - 0.31
B) Tubular	• Produces high shear and entrainment as water-air mixture is forced through vertical cylinder containing static mixing elements. • Cylinder construction is metal or plastic.	• Primarily aerated lagoon applications.	• Economically attractive. • Low maintenance. • High transfer efficiencies for diffused air systems. • Well suited for aerated lagoon applications.	• Ability to adequately mix reactor basin contents is questionable. • Application for use in high-rate biological systems unconfirmed.	0.31 - 0.44
C) Jet	• Compressed air and pumped liquid are violently intermixed in nozzle and at discharge into vessel.	• Same as for bubbler diffuser.	• Suited for deep tanks. • Moderate cost.	• Tank geometry limited. • Clogging of nozzle requires blower and pump. • Primary treatment required.	0.43 - 0.60
Mechanical Surface: A) Radial flow, low speed 20 - 60 RPM	• Low output speed, large diameter turbine, floating, fixed bridge or platform mounted. • Used with gear reducer.	• Same as for bubbler diffuser.	• Tank design flexibility. • High pumping capacity.	• Some icing in cold climates. • Initial cost higher than axial flow aerators. • Gear reducer may cause maintenance problems.	0.34 - 0.76
B) Axial flow 300 - 1,200 RPM	• High output speed. Small diameter propeller. • They are direct, motor driven units mounted on floating structures.	• Aerated lagoons and reaeration.	• Low initial cost. • Easy to adjust to varying water level flexible operation.	• Some icing in cold climates. • Poor maintenance accessibility. • Mixing capacity may be inadequate.	0.34 - 0.43
C) Brush rotor	• Low output speed. • Used with gear reducer.	• Oxidation ditch, applied either as an aerated lagoon or as an activated sludge.	• Moderate initial cost. • Good maintenance accessibility.	• Subject to operational variables which may affect efficiency. • Tank geometry is limited.	0.43 - 0.60
Submerged turbine	• Units contain a low-speed turbine and provide compressed air to diffuser rings or open pipe. • Fixed-bridge application.	• Same as for bubbler diffuser.	• Good mixing, high-capacity input per unit volume. • Deep tank application. • Operational flexibility. • No icing or splash.	• Require both gear reducer and blower. • High total power requirements. • High cost.	0.29 - 0.43

*Reported efficiency varies because of tank geometry, design, and other factors.

Air piping systems should be designed such that total head loss from the blower outlet (or silencer outlet where used) to the diffuser inlet does not exceed 3.4 kPa (0.5 psi) at average operating conditions.

The spacing of diffusers should be in accordance with the oxygen requirements through the length of the channel or tank and should be designed to facilitate adjustment of their spacing without major revision to air header piping. Fifty percent (50%) blanks should be provided in at least the first half of the aeration system, for possible addition of more diffusers if found necessary.

All plants should be designed to incorporate removable diffusers that can be serviced and/or replaced without dewatering the tank.

- Individual assembly units of diffusers should be equipped with control valves, preferably with indicator markings for throttling, or for complete shut-off. Diffusers in any single assembly should have substantially uniform pressure loss.
- Air filters should be provided in numbers, arrangements, and capacities to always furnish an air supply sufficiently free from dust to prevent damage to blowers and clogging of the diffuser system used. Blowers must have silencers, flexible connections, and gauges.
- Blowers which require internal lubrication are not desirable because of the danger of diffuser clogging from oil being carried in the air stream. Water-piston-type compressors are not desirable because they increase the condensation in the air system, resulting in a more severe corrosion problem in the piping and greater pressure loss required to pass the condensate through the diffusers.

8.4.1.2.8 Mechanical Aeration Systems

Oxygen Transfer Performance

The mechanism and drive unit should be designed for the expected conditions in the aeration tank in terms of the power performance. Certified testing should verify mechanical aerator performance. In the absence of specific design information, the oxygen requirements should be calculated using a transfer rate not to exceed 1.22 kg O_2/kW·h in clean water under standard conditions.

Design Requirements

- Maintain a minimum of 2.0 mg/L DO in the mixed liquor at all times throughout the tank or basin.
- Maintain all biological solids in suspension.
- Meet maximum oxygen demand and maintain process performance with the largest unit out of service.
- Provide for varying the amount of oxygen transferred in proportion to the load demand on the plant.
- Provide that motors, gear housing, bearings, grease fittings, etc. be easily accessible and protected from inundation and spray as necessary for the proper functioning of the unit.
- Winter protection.

Due to high heat loss, the mechanism, as well as subsequent treatment units, should be protected from freezing and other considerations for climate and climate change impacts where extended cold weather conditions occur.

8.5 Rotating Biological Contactors

8.5.1 General

8.5.1.1 Applicability

The RBC process may be used where wastewater is amenable to biological treatment. The process may be used to accomplish carbonaceous and/or nitrogenous oxygen demand reductions.

Considerations for the RBC process should include:

- Raw wastewater amenability to biological treatment.
- Pretreatment effectiveness including scum and grease removal.
- Expected organic loadings, including variations.
- Expected hydraulic loadings, including variations.
- Treatment requirements, including necessary reduction of carbonaceous and/or nitrogenous oxygen demand.
- Wastewater characteristics (including pH, temperature, toxicity, and nutrients).
- Maximum organic loading rate of active disc surface area.
- Minimum detention time at maximum design flow.
- Low whole life costs.

8.5.1.2 Winter Protection (Enclosures)

Wastewater temperature affects rotating contactor performance. Year-round operation requires that rotating contactors be covered to protect the biological growth from cold temperatures and the excessive loss of heat from the wastewater with the resulting loss of performance. Construction of enclosures should take climate and climate change impacts into consideration (e.g., insulation from cold weather, locking/securing mechanisms for high winds, and/or enclosure flanges or building joints at a level to prevent water ingress). Inclusion of insulated covers should not make manual handling tasks impractical, where removal of covers is carried out for inspection services.

Enclosures should be constructed of a suitable corrosion resistant material. Windows or simple louvered mechanisms which can be opened in the summer and closed in the winter should be installed to provide adequate ventilation. To minimize condensation, the enclosure should be adequately insulated and/or heated. Mechanical ventilation should be supplied when the RBCs are contained within a building provided with interior access for personnel.

8.5.1.3 Required Pre-Treatment

Rotating biological contactors are dependent on the preceding treatment steps to effectively reduce the solids or BOD_5 level in high-strength influent streams that could otherwise result in interference or overload. Efficient grit removal and screening are, therefore, essential to prevent solids build-up under the filter media. Effective primary treatment processes must be provided to reduce the organic and hydrogen sulphide loading to the RBC unit, unless substantial justification is submitted for other pre-treatment devices which provide for effective removal or grit, debris and excessive oil or grease prior to the RBC units. Bar screening or comminution are not sole means of pre-treatment.

8.5.1.4 Flow Equalization

For economy of scale, the peaking factor of maximum flow to ADF should not exceed 3.0. Flow equalization should be considered in any instance where the peaking factor exceeds 2.5.

8.5.1.5 Operating Temperature

The temperature of wastewater entering any RBC should not drop below 13°C unless there is sufficient flexibility to decrease the HLR, or the units have been increased in size to accommodate the lower temperature. Otherwise, insulation or additional heating must be provided to the plant.

8.5.1.6 Design Flexibility

Adequate flexibility in process operation should be provided by considering one or more of the following:

- Variable rotational speeds in first and second stages.
- Multiple treatment trains.
- Removable baffles between all stages.
- Positive influent flow control to each unit or flow train.
- Positively controlled alternate flow distribution systems.
- Positive airflow metering and control to each shaft when supplemental operation or air drive units are used.
- Recirculation of secondary clarifier effluent.

8.5.1.7 Hydrogen Sulphide

When higher than normal influent or side stream hydrogen sulphide concentrations are anticipated, appropriate modifications in the design should be made.

8.5.2 Unit Sizing

The designer of an RBC system should conform to the following design criteria unless it can be shown by thorough documentation that other values or procedures are appropriate. This documentation may include detailed design calculations, pilot test results, and/or manufacturer’s empirical design procedures. It should be noted that use of manufacturer’s design procedures should be tempered with the realization that they are not always accurate and, in some cases, can substantially overestimate attainable removals.

8.5.2.1 Unit Sizing Considerations

Unit sizing should be based on experience at similar full-scale installations or thoroughly documented pilot testing with the particular wastewater. In determining design loading rates, expressed in units of volume per day per unit area of media covered by biological growth, the following parameters must be considered:

- Design flowrate and influent waste strength.
- Percentage of BOD_5 to be removed.
- Media arrangement, including number of stages and unit area in each stage.
- Rotational velocity of the media.
- Retention time within the tank containing the media.
- Wastewater temperature.
- Percentage of influent BOD_5 which is soluble.

In addition to the above parameters, loading rates for nitrification will depend upon influent TKN, pH, and allowable effluent ammonia nitrogen concentration.

8.5.2.2 Hydraulic Loading

Hydraulic loading to the RBCs should range between 75 to 155 L/m^2d of media surface area without nitrification, and 30 to 80 L/m^2d with nitrification.

8.5.2.3 Organic Loading

The RBC process is approximately first order with respect to BOD_5 removal (i.e., for a given hydraulic loading or retention time) a specific percent BOD_5 reduction will occur, regardless of the influent BOD_5 concentration, however, BOD_5 concentration does have a moderate effect on the degree of treatment, and thus the possibility of organic overloading in the first stage. With this in mind, organic loading to the first stage of an RBC train should not exceed 0.03 to 0.04 kg BOD/m^2d or 0.012 to 0.02 kg BOD_5 $soluble/m^2d$.

Loadings in the higher end of these ranges will increase the likelihood of developing problems such as heavier than normal biofilm thickness, depletion of DO, nuisance organisms, and deterioration of overall process performance. The structural capacity of the shaft, provisions for stripping biomass, consistently low influent levels of sulphur compounds to the RBC units, the media surface area required in the remaining stages, and the ability to vary the operational mode of the facility may justify choosing a loading in the high end of the range, but the operator must carefully monitor process operations.

Under normal operation carbonaceous substrate is mainly removed in the initial stage of the RBC, therefore, the design organic loading rate applied to a RBC system particularly on the first stage must be within the oxygen transfer capability of the system to avoid overloading, odours, sloughing problems, and the development of nuisance bacteria.

8.5.2.4 Tank Volume

For purposes of plant design, the optimum tank volume is measured as wastewater volume held within a tank containing a shaft of media per unit of growth covered surface on the shaft, or L/m^2. The optimum tank volume determined when treating domestic wastewater up to 300 mg/L BOD_5 is 4.9 L/m^2, which considers wastewater displaced by the media and attached biomass. The use of tank volumes in excess of 4.9 L/m^2 does not yield corresponding increases in treatment capacity when treating wastewater in this concentration range.

8.5.2.5 Detention Time

Based on a tank volume of 4.9 L/m^2, the detention time in each RBC stage should range between 40 and 120 minutes without nitrification, and 90 and 250 minutes with nitrification.

8.5.2.6 Media Submergence & Clearance

Rotating biological contactors should operate at a submergence of approximately 40% based on total media surface area. To avoid possible shaft overstressing and inadequate media wetting, the liquid operating level should never drop below 35% submergence. Media submergence of up to 95% may be allowed if supplemental air is provided. A clearance of 10 to 23 cm. between the tank floor and the bottom of the rotating media should be provided to maintain sufficient bottom velocities to prevent solids deposition in the tank.

8.5.3 Design Considerations

8.5.3.1 Unit Staging

The arrangement of media in a series of stages has been shown to significantly increase treatment efficiency. It is, therefore, recommended that an RBC plant be constructed in at least four stages for each flow path (or four zones of media area).

Four stages may be provided on a single unit by providing baffles within the tank. For small installations where the total area requirements dictate two units per flow path, two units may be placed in series with a single baffle in each tank, thus providing the minimum of four stages. For larger installations requiring four or more units per flow path, the units may be placed in a series within the flow path, with each unit itself serving as a single stage. Generally, though, plants requiring more than four stages should be constructed in a series of parallel floor trains, each comprised of four separate stages.

Wastewater flow to RBC units may be either perpendicular or parallel to the media shafts.

8.5.3.2 Tankage

Rotating biological contactor units may be placed in either steel, Fiber Reinforced Plastic (FRP), or concrete tankage with baffles when required, and constructed of a variety of materials. The design of the tankage must include:

- Adequate structural support for the RBC and drive unit.
- Elimination of the “dead” areas.
- Satisfactory hydraulic transfer capacity between stages of units.
- Considerations for operator safety.

The structure should be designed to withstand the increased loads which could result if the tank were to be suddenly dewatered with a full biological growth on the RBC units. The sudden loss of buoyancy resulting from unexpected tank dewatering could increase the bearing support loadings by as much as 40%.

Provisions for operator protection can be included in the tankage design by setting the top of the RBC tankage about 1 ft above the surrounding floor and walkways, with handrails placed along the top of the tankage, to provide an effective barrier between the operator and exposed moving equipment. The high tank walls will also prevent loss or damage by any material accidentally dropped in the vicinity of the units and entering the tankage.

8.5.3.3 Rotating Batch Contactor Media

The RBC media should be manufactured from Polypropylene Copolymer (PP) or HDPE sheets and stabilised against UV decay. The media sheet material should be selected to provide dimensional stability, high tear resistance and tensile strength at the operational temperature range during storage and operation.

Disc media may be supplied in different sheet configurations or corrugation patterns. Corrugations are preferred as this increases the available surface area and enhance their structural stability.

The media sheets may be cut into wedge shaped segments to facilitate ease of maintenance. Alternative sheet arrangements must facilitate ease of maintenance without the requirement to remove the RBC shaft.

Waterways and airways within and between the media sheets must be maintained under all conditions and be of sufficient size to prevent bridging of the biomass and to allow oxygenation. The RBC media should be arranged in such a manner that the resulting specific media substrate loading rate does not cause the biomass to block the media by bridging the gap between the media sheets.

The thickness and weight of biomass should be based on the maximum design substrate-loading rate on the RBC media.

The media sheet should be designed to support the static and dynamic loads resulting from the weight of the media sheet and attached biomass and should take account of the rotation through water and air.

8.5.3.4 High Density Media

Except under special circumstances, high density media should not be used in the first stage. Its use in subsequent stages should be based on appropriate loading criteria, structural limitations of the shaft and media, and media configuration.

8.5.3.5 Shaft Rotational Speed

The shafts are rotated (1-to-2 revolutions per minute) by either mechanical or compressed air drive. Provision should also be made for rotational speed control and reversal.

8.5.3.6 Biomass Removal

A means for removing excess biofilm growth should be provided, such as air or water stripping, chemical additives, etc.

8.5.3.7 Dissolved Oxygen Monitoring

First-stage DO monitoring should be provided. The RBC should be able to maintain a positive DO level in all stages.

8.5.3.8 Supplemental Air

Periodic high organic loadings may require supplemental aeration in the first stage to promote sloughing of biomass.

8.5.3.9 Side Stream Inflows

The type and nature of side stream discharges to an RBC must be evaluated, and the resulting loads must be added to the total plant influent loads. Anaerobic digesters increase ammonia nitrogen loadings, and sludge conditioning processes such as heat treatment contribute increased organic and ammonia nitrogen loadings. Whenever septic tank discharges comprise part of the influent wastewater or any unit processes are employed that may produce sulphide ahead of the RBC units, the additional oxygen demand associated with sulphide must be considered in system design.

8.5.3.10 Recirculation

For small installations, such as those serving an industrial park or school, the inflow over weekends or at holiday periods may drop to zero. During such periods, the lack of incoming organic load will cause the media bio-growth to enter the endogenous respiration phase where portions of the bio-growth become the food source or substrate for other portions of the bio-growth. If this condition lasts long enough, all of the bio-growth will eventually be destroyed. When this condition is allowed to exist, the RBC process does not have adequate bio-growth to provide the desired treatment when the inflow restarts.

If flow can be recycled through the sludge holding/treatment units and then to the RBC process, an organic load from the sludge units can be imposed on the RBC process. This imposed load will help to maintain the bio-growth and, as a secondary benefit, help stabilize and reduce the sludge.

When any new plant is first started, the bio-growth is slow to establish. If it is desired to build up the bio-growth before directing all of the inflow to the RBC process (as when the RBC is replacing an older existing process) some inflow may be directed to the RBC process and recycled.

In the first few days, minimal bio-growth will develop with only minimal removal of the organic load. By recycling, the unused organic load again becomes available to the bio-growth. As the bio-growth develops, the recycle rate should be reduced, with new inflow added to increase the organic load. As the bio-growth develops further, the recycle is eventually reduced to zero with all of the inflow being the normal RBC influent.

Where recirculation is provided:

- The recirculation system and its control should be designed to reduce the risk of anaerobic conditions being established, which would impair the efficiency of the plant.
- Each RBC unit or all the RBC streams together should contain a recirculation pumping system capable of providing a recirculation (by returning RBC outlet liquor back to the RBC inlet). Where a common recirculation pumping system is provided, then consideration should be given to a flow through pumping station located downstream of the final effluent sample chamber.

8.5.3.11 Load Cells

Load cells, especially in the first stage(s), can provide useful operating and shaft load data. Where parallel trains are in operation, they can pinpoint overloaded or under loaded trains. Torque switch, Loss of Rotation (LOR) indicators, and clamp-on ammeters are also potentially useful monitoring instruments.

Load cells should be provided for all first and second stage shafts. Load cells for all other shafts in an installation are desirable.

8.5.3.12 Loss of Rotation

Each RBC shaft should be fitted with a LOR warning device which must be configured with an operator adjustable (0 to 5 minutes) time delayed alarm to indicate a loss of shaft rotation under any circumstance, other than a failure in the power supply. The LOR alarm system must stop the RBC motor and generate an alarm signal at the Motor Control Centre (MCC).

8.5.3.13 Shaft Access

In all RBC designs, access to individual shafts for repair or possible removal must be considered. Bearings should also be accessible for easy removal and replacement if necessary. Where all units in a large installation are physically located very close together, it may be necessary to utilize large off-the-road cranes for shaft removal. Crane reach, crane size, and the impact of being able to drain RBC tankage and dry a unit prior to shaft removal should all be considered when designing the RBC layout.

8.5.3.14 Structural Design

The designer should require the manufacturer to provide adequate assurance that the shaft and media support structures are protected from structural failure for the design life of the plant. Structural designs should be based on appropriate American Welding Society (AWS) stress category curves modified as necessary to account for the expected corrosive environment. All fabrication during construction should conform to AWS welding and quality control standards.

Loads for which the shaft structure is designed should be determined based on the biomass weight and weight of the structural steelwork as follows:

- Biomass thickness and the resulting weight should be based on the maximum design substrate-loading rate applied to the RBC media.
- Dead weight of the RBC structure should include static weight of the shaft tube, media support structure, media, and fixings.

8.5.3.15 Geared Motors

A high efficiency helical-helical or helical-bevel shaft mounted close coupled geared motor should be supplied to drive each RBC. The gearbox should be clamped to the stub shaft by a shrink disc coupling. The drive unit should be attached to the support frame with a torque reaction arm. A minimum service factor of two should be applied to the gearbox selection. This service factor should be based on the estimated torque demand for continuous duty at the selected specific media substrate-loading rate (kg $BOD_5/m^2/d$) and the potential out of balance load resulting from prolonged stoppage of the RBC due to mechanical or electrical failure or interruption in power supply.

The RBC drive system should incorporate a soft start starter with an adjustable ramp up time set to enable the lowest part of RBC to go past the “top-dead centre” when the RBC is started. The RBC drive system should be capable of re-starting the RBC after a power failure and stoppage of 8 hours without the need to rebalance the RBC for dynamic loads (i.e., a pulsed restart system).

There should be sufficient space around the gearbox and motor to enable O&M personnel to carry out their tasks safely.

8.5.3.16 Energy Requirements

Energy estimates used for planning and design should be based on expected operating conditions such as temperature, biofilm thickness, rotational speed, type of unit (either mechanical or air driven), and media surface area instead of normalized energy data sometimes supplied by equipment manufacturers. Care should be taken to assure that manufacturer’s data are current and reflect actual field-validated energy usage.

Only high efficiency motors and drive equipment should be specified. The designer should also carefully consider providing power factor correction for all RBC units.

8.5.3.17 Nitrification Consideration

Effluent concentrations of ammonia nitrogen from the RBC process designed for nitrification are affected by diurnal load variations, therefore, it may be necessary to increase the design surface area proportional to the ammonia nitrogen diurnal peaking rates to meet effluent limitations. An alternative is to provide flow equalization sufficient to insure process performance within the required effluent limitations.

In addition to the above parameters, loading rates for nitrification, when required, should depend upon influent TKN, pH, and allowable effluent ammonia nitrogen concentration.

8.5.3.18 Maintenance

All RBC equipment should, at minimum, meet the following maintenance requirements:

- All regular maintenance should be achievable without requiring access below chamber covers.
- No elements that require greasing should be below the water line. Extension hoses, with caps, for grease nipples to be provided to minimize to the extent possible the need for operators to enter the enclosure.
- Minimum clearance should be provided such that parts can be removed in a safe and reliable way, without interference to operation.

8.6 Waste Stabilization Ponds

8.6.1 Supplement to Pre-Design Report

8.6.1.1 General

The Pre-design Report should contain pertinent information on location, geology, soil conditions, area for expansion, and any other factors that will affect the feasibility and acceptability of the proposed project.

The following information must be submitted in addition to that required in Chapter 1.

8.6.1.2 Location in Relation to Nearby Facilities

The location and direction of all residences, commercial developments, recreational areas, and water supplies within 2 km of the proposed pond should be included in the Pre-design Report.

8.6.1.3 Land Use Zoning

Land use zoning adjacent to the proposed pond site should be included.

8.6.1.4 Soil Borings

Data from soil borings, conducted by an independent soil testing laboratory to determine subsurface soil characteristics and groundwater characteristics (including elevation and flow) of the proposed site and their effect on the construction and operation of a pond, should also be provided. At least one boring should be a minimum of 7.5 m in depth or into bedrock, whichever is shallower. If bedrock is encountered, rock type, structure and corresponding geological formation data should be provided. The boring should be filled and sealed. The permeability characteristics of the pond bottom and pond seal materials should also be studied.

8.6.1.5 Percolation Rates

Data demonstrating anticipated percolation rates at the elevation of the proposed pond bottom should be included. In-situ permeability testing should be done to measure the percolation rates.

8.6.1.6 Site Description

A description, including maps showing elevations and contours of the site and adjacent area suitable for expansion should be identified. Due consideration should be given to additional treatment units and/or increased waste loadings or determining load requirements.

8.6.1.7 Location of Field Tile

The location, depth, and discharge point of any field tile (subsurface drainage systems) in the immediate area of the proposed site should be identified so that proper separation distances from proposed facilities can be maintained.

8.6.1.8 Sulphate Content of Water Supply

Sulphate content of the basic water supply should be determined.

8.6.1.9 Well Survey

A pre-construction survey of all nearby wells (water level and water quality) is mandatory.

8.6.2 Location

8.6.2.1 Distance from Habitation

For separation distances, see Section 5.3.2.1.

8.6.2.2 Prevailing Winds

If practical, ponds should be located so that local prevailing winds will be in the direction of uninhabited areas.

8.6.2.3 Surface Runoff

Location of ponds in watersheds receiving significant amounts of storm water runoff is discouraged. Adequate provision must be made to divert storm water runoff around the ponds and protect pond embankments from erosion. The impacts of climate change (e.g., potential increases in the quantity of surface water runoff and intensity of storm events leading to increased erosion) should be considered when selecting location and design parameters.

8.6.2.4 Groundwater Pollution

Existing wells which serve as drinking water sources should be protected from health hazards or as required by the Regulatory Authority. Possible travel of pollutants through porous soils and fissured rocks should be objectively evaluated to safeguard the wells. A pond should be located as far as practical, with a minimum of 300 m from any well used as a drinking water source.

A minimum separation of 1.2 m between the bottom of the pond and the maximum groundwater elevation should be maintained, however, less separation may be acceptable when supported by appropriate hydrogeological and engineering designs/investigations upon acceptance of the Regulatory Authority having jurisdiction. Considerations for the impacts of climate change on the maximum groundwater elevation should be accounted for.

A minimum separation of 1.5 m between the bottom of the pond and bedrock is recommended, however, less separation may be acceptable when supported by appropriate hydrogeological and engineering designs/investigations upon acceptance of the Regulatory Authority having jurisdiction.

8.6.2.5 Protection of Surface Water Supplies

Stabilization basins should be located downhill, downstream, and remote from all sources of surface water drinking supplies (lakes and rivers). The minimum distances listed in Table 8.5 should be employed as the criteria, but should consider climate change (i.e., in cases where climate change impacts the surface extent of lakes or rivers).

Table 8.5: Surface Water Separation Distances

Minimum Distance from a Lake or River to the Centre of a Dyke of a Proposed Stabilization Basin	Remarks
120 m	Lined stabilization basin, pervious soil.
75 m	Lined stabilization basin, impervious soil.

8.6.2.6 Geology

Ponds should not be located in areas which may be subjected to karstification (i.e., sink holes or underground streams generally occurring in areas underlain by limestone or dolomite).

8.6.2.7 Floodplains

A pond should not be located within the 1-in-100-year floodplain including climate change considerations and design considerations should be made to minimize damages from the impacts of storm surge including climate change considerations (e.g., sea level rise and increased storm intensity) in coastal areas.

8.6.3 Definitions

8.6.3.1 Aerobic Stabilization Basins

Aerobic lagoons are shallow basins which use natural processes involving both algae and bacteria. Aerobic lagoons have a minimum depth of 1 m. Oxygen is provided by algae during photosynthesis and wind aided surface aeration. The physical dimensions, temperature, amount of sunlight, and amount of natural or artificial turbulence are used to maintain a desired DO concentration. In practice it is not possible to maintain a completely aerobic lagoon. The bottom sediments will contain some facultative bacteria.

8.6.3.2 Facultative Stabilization Basins

Facultative lagoons are the most common type and are also referred to as oxidation lagoons. These lagoons are typically 1.5 m deep, with detention times ranging from 25 to more than 180 days. Depths are kept at 1.5 m or more to avoid the growth of emergent plants. Surface layers of the lagoon are aerobic with an anaerobic layer near the bottom. Oxygen is supplied by surface aeration and photosynthetic algae. Facultative lagoons are designed in series with a minimum of three cells to reduce short circuiting. The primary problem with facultative lagoons is the production of algae that remains in the effluent, which sometimes causes effluent SS to exceed discharge requirements.

8.6.3.3 Aerated Stabilization Basins

Aerated lagoons can be either partially mixed or completely mixed. Oxygen is supplied by mechanical floating aerators or diffused aeration. Aerated lagoons have a minimum depth of 3 m, with detention times ranging from 5 to 30 days. Aerated lagoons accept higher BOD_5 loadings than facultative lagoons, are less susceptible to odours, and typically require less land. Aerated lagoons are followed by a facultative lagoon or a settling lagoon (1-day detention or less) to reduce SS before discharge.

8.6.3.4 Anaerobic Stabilization Basins

Anaerobic lagoons are heavily loaded with organics and do not have an aerobic zone. They have a minimum depth of 3.0 m and detention times of 20 to 50 days. Biological activity is typically low when compared to that of a mixed anaerobic digester. Anaerobic lagoons have been used as pre-treatment to facultative and aerobic lagoons for strong industrial wastewater and for rural communities with a significant organic load from industries such as food processing.

8.6.4 Application, Advantages, & Disadvantages of Different Stabilization Basin Types

Table 8.6 presents advantages and disadvantages of the different pond and stabilization basin types for various applications.

Table 8.6: Application, Advantages, & Disadvantages of the Different Stabilization Basin Types

Parameter	Unaerated Aerobic	Facultative	Anaerobic	Aerated	
				Aerobic	Facultative
Application	Nutrient removal, treatment of soluble organic wastes and secondary effluents.	Treatment of raw domestic and industrial wastes.	Pretreatment for strong industrial wastewater and areas with large organic load.	Treatment of raw domestic and industrial wastes.	Treatment of raw domestic and industrial wastes.
Advantages	Low O&M costs.	Low O&M costs.	Small volume and area, effective for high strength wastewater.	Small volume and area, resistance to upsets.	Small volume and area, resistance to upsets.
Disadvantages	Large volume and area, possible odours.	Large volume and area, possible odours.	Significant maintenance, gas safety, possible odours.	Significant O&M costs, high solids in effluent, foaming.	O&M costs, foaming.

8.6.5 Basis of Design

8.6.5.1 Waste Stabilization Basins

8.6.5.1.1 Holding Capacity Requirements

Before the design of a waste stabilization pond system can be initiated, the designer should determine the following:

- Whether the stabilization basin can be continuously discharged or operate on a fill-and-draw basis (intermittent discharge).
- The period of the year if any when discharge will not be permitted.
- What discharge rates will be permitted with fill-and-draw stabilization basin and what, if any, provision must be made for controlling effluent discharge rates in proportion to receiving stream flowrates.
- What the minimum time for discharge of stabilization basin cell contents should be for fill-and-draw systems.

The holding capacity of ponds should be based upon average daily wastewater flowrates, making a special allowance for net precipitation entering the cells.

8.6.5.1.2 Area & Loadings

One (1) ha of water surface should be provided for each 250 design population or PE. In terms of BOD_5, a loading of 22 kg BOD_5/ha·day should not be exceeded. Higher or lower design loadings will be judged after review of material contained in the Pre-design Report and after a field investigation of the proposed site by the Regulatory Authority having jurisdiction.

Due consideration should be given to possible future municipal expansion, climate change impacts, and/or additional sources of wastes when the original land acquisition is made. Suitable land should be available at the site for increasing the size of the original construction without sacrificing recommended separation distances.

Where substantial ice cover may be expected for an extended period, it may be desirable to operate the plant to completely retain winter-time flows.

Design variables such as pond depth, multiple units, detention time and additional treatment units must be considered with respect to applicable standards for BOD_5, TSS, fecal coliforms, DO, and pH.

8.6.5.1.3 Flow Distribution

The main inlet sewer or force main should terminate at a chamber which permits hydraulic and organic load splitting between the stabilization basin cells. The ability to introduce raw wastewater to all cells is desirable but as a minimum, there must be a capability to divide raw wastewater flows between enough cells to reduce the BOD_5 loading to 22 kg BOD_5/ha·day, or less. Complete isolation of each cell should be allowed for, for maintenance and inspection.

The inlet chamber should be provided with a lockable aluminum cover plate or grating, divided into small enough sections to permit easy handling.

8.6.5.1.4 Controlled Discharge Stabilization Ponds

For controlled-discharge systems, the area specified as the primary ponds should be equally divided into two cells. The third or secondary cell volume should, as a minimum, be equal to the volume of each of the primary cells.

In addition, the design should permit for adequate elevation difference between primary and secondary ponds to permit gravity filling of the secondary from the primary. Where this is not feasible, pumping facilities may be provided.

8.6.5.1.5 Flow-Through Pond Systems

At a minimum, primary cells should provide adequate detention time to maximize BOD_5 removal. Secondary cells should then be provided for additional detention time with depths to 2.0 m to facilitate both solids and pathogen reduction.

8.6.5.1.6 Tertiary Pond

When ponds are used to provide additional treatment for effluents from existing or new secondary wastewater treatment works, the *Clean Water Act Analytical Methods* from the United States Environment Protection

Agency will, upon request, establish BOD_5 loadings for the pond after due consideration of the efficiencies of the preceding treatment units.

8.6.5.2 Aerated Stabilization Basins

8.6.5.2.1 General

Aerated ponds can be either aerobic or facultative. An aerated aerobic pond contains DO through the whole system with no anaerobic zones. The pond shape and the aerating power provides complete mixing. The aerated facultative pond provides a partially mixed condition which will cause an anaerobic zone to develop at the bottom as SS settle due to low velocity in the system.

Aerated Aerobic Stabilization Basins

In general, an aerated pond can be classified as an aerobic pond (complete mixed) if the mechanical aeration power level is above 6 W/m^3 of maximum storage.

Aerated aerobic ponds should be designed to maintain complete mixing with bottom velocities of at least 0.15 m/s. It is important that sufficient mixing power be provided.

Quiescent settling areas adjacent to the aerated cell outlets or the addition of SS removal processes such as a clarifier must follow aerated aerobic treatment, to ensure compliance with SS discharge requirements. In most cases, a minimum detention time of 1 day is required to achieve solids separation. Algae growth should be limited by controlling the hydraulic detention time to 2 days or less. Water depth of not less than 1.0 m should be maintained to control odours arising from anaerobic decomposition. Adequate provision must be made for sludge storage so that the accumulated solids will not reduce the actual detention time.

Aerated Facultative Stabilization Basins

Aerated facultative ponds should be designed to maintain a minimum of 2.0 mg/L of DO in the upper zone of the liquid.

The aeration system must be able to transfer up to 1.0 kg O^2/kg BOD_5 applied uniformly throughout the pond when the water temperature is 20°C. The organic loading rate should be maintained between 0.031 and 0.048 $kg/m^3 \cdot day$.

The escape of algae into the effluent should be controlled by providing a quiescent area adjacent to each cell outlet with an overflow rate of 32 $m^3/m^2 \cdot d$. If multiple aerated facultative cells are used, all cells following the first one should have diminished aeration capacity to permit additional settling.

Whenever possible, provisions should be provided for recirculating part (5 to 10%) of the final aeration cell effluent back into the influent to maintain a satisfactory mix of active micro-organisms.

8.6.5.2.2 Design Approach

In general, the following factors should be considered in the design of the aerated lagoons:

- 5-day biochemical oxygen demand removal and effluent characteristic.
- Temperature effects.
- Mixing requirements.

- Oxygen requirements.
- Solids separation.

8.6.5.2.3 BOD_5 Removal and Effluent Characteristic

Biochemical oxygen demand removal and the effluent characteristics are estimated using a complete mix hydraulic model and first order reaction kinetics.

The complete mixed model using first order kinetics and operating in a series with 'n' equal volume cells is given by:

$$\frac{L_e}{L_i} = \frac{1}{\left[1 + \frac{K_t T}{n}\right]^n}$$

Where:

L_e	=	Effluent BOD_5 (mg/L).
L_i	=	Influent BOD_5 (mg/L).
K_t	=	Reaction rate coefficient at t°C (day^{-1}).
T	=	Total HRT in lagoon system (days).
n	=	Number of cells in series.

The selection of the reaction rate coefficient is critical in the design of the lagoon system. All other considerations in the design will be influenced by this selection. If possible, a design K_{20} should be determined for the wastewater in pilot or bench scale tests; experiences of others with similar wastewaters and environmental conditions should also be evaluated. Reaction rate coefficient K_{20} may vary from 0.276 day^{-1} for complete mix cell to 0.138 day^{-1} for aerated cell.

When using the complete mix model, the number of cells in series has a pronounced effect on the size of the aerated cell required to achieve a specific degree of treatment. The reactor required to achieve a given efficiency may be greatly reduced by increasing the number of cells in series.

8.6.5.2.4 Temperature Effects

The influence of temperature on the reaction rate is expressed by the following equation:

$$K_t = K_{20}\ \theta^{t-20}$$

Where:

K_t	=	Reaction rate coefficient at t°C (day^{-1}).
K_{20}	=	Reaction rate coefficient at 20°C (day^{-1}).
t	=	Wastewater temperature (°C).
θ	=	Temperature activity coefficient. (varies from 1.04 to 1.1 for aerated lagoons, with typical value of 1.035).

Considerations for the impact of climate change on design temperatures should be accounted for and the most conservative value selected for design.

8.6.5.2.5 Oxygen Requirements

Oxygen requirements generally will depend on the BOD_5 loading, the degree of treatment and the concentration of SS to be maintained. Aeration equipment should be capable of maintaining a minimum DO level of 2.0 mg/L in the ponds at all times.

The oxygen requirements should meet or exceed the peak 24-hour summer loadings. A safety factor of up to two should be considered in designing oxygen supply equipment based on average BOD_5 loadings. The amount of oxygen requirement has been found to vary from 0.7 to 1.5 times the amount of BOD_5 removed.

8.6.5.2.6 Mixing Requirements

Aeration is used to mix the pond contents and to transfer oxygen to the liquid. There is no rational method available to predict the power input necessary to keep the solids suspended. The best approach is to consult equipment manufacturer's charts and tables to determine the power input needed to satisfy mixing requirements. Power of 6 to 10 W/m^3 of the cell volume is frequently used for complete mix design arrangements, and these values can be used as a guide to make preliminary estimates of power requirements, but the final sizing of aeration equipment should be based on guaranteed performance by an equipment manufacturer.

For a complete mix cell, in comparing the power requirements for both, to maintain solids in suspension and to meet the oxygen demand, it would soon become evident that the mixing requirements would control the power input to the system.

After determining the total power requirements for a cell, the diffusers/aeration units should be located in the cell so that there is an overlap of the diameter of influence providing complete mixing.

8.6.5.2.7 Solids Separation

For systems with continuous discharge to a receiving stream, a polishing cell having a minimum hydraulic retention of 5 days, based on summer average daily design flows, should be provided. Polishing cells are not required for systems having storage facilities with intermittent discharges.

8.6.5.3 Industrial Wastes

Due consideration should be given to the type and effects of industrial wastes on the treatment process. In some cases, it may be necessary to pre-treat industrial or other discharges.

8.6.5.4 Multiple Units

At a minimum, a waste stabilization pond system should consist of two cells designed to facilitate both series and parallel operations. The maximum size of a pond cell should be 5.0 ha. A one cell system may be utilized in small installations. Larger cells may be permitted for bigger installations.

All systems should be designed with piping flexibility to permit isolation of any cell without affecting the transfer and discharge capabilities of the total system.

Requirements for multiple units in an aerated stabilization basin system should be similar to those in an activated sludge system, including requirements for back-up aeration equipment.

8.6.5.5 Design Depth

The minimum operating depth should be sufficient to prevent growth of aquatic plants and damage to the dykes, control structures, aeration equipment and other appurtenances. See Section 8.6.1 for typical pond depths.

8.6.5.6 Pond Shape

Acute angles within any wastewater stabilization pond or aerated stabilization basin should be avoided. Square cells are preferred to long narrow rectangular cells. The long dimension of any pond should not align with the prevailing wind direction.

8.6.6 Pond Construction Details

8.6.6.1 Embankments & Dykes

8.6.6.1.1 Materials

Embankments and dykes should be constructed of relatively impervious materials and compacted to at least 95% Standard Proctor Density to form a stable structure. Vegetation and other unsuitable materials should be removed from the area where the embankment is to be placed.

A soils consultant's report should be required for all earthen berm construction to demonstrate the suitability of the soils. In certain instances, a hydrogeologist's report may be required to assess possible impact on the water table. All topsoil must be stripped from the area on which the berms are to be constructed.

8.6.6.1.2 Top Width

The minimum embankment top width should be 3.0 m to permit access of maintenance vehicles.

8.6.6.1.3 Maximum Slopes

Unless otherwise specified by a soil consultant's report, embankment slopes should not be steeper than:

Inner

Three (3) horizontal to one vertical (3_H:1_V).

Outer

Three (3) horizontal to one vertical (3_H:1_V).

8.6.6.1.4 Minimum Slopes

Embankment slopes should not be flatter than:

Inner

Four (4) horizontal to one vertical (4_H:1_V). Flatter slopes are sometimes specified for larger installations because of wave action but have the disadvantage of added shallow areas conducive to emergent vegetation. Other methods of controlling wave action may be considered.

Outer

Outer slopes should be sufficient to prevent surface water runoff from entering the ponds.

8.6.6.1.5 Freeboard

Minimum freeboard should be 1.0 m, or as required based on water level and runoff projections including climate change considerations (e.g., potential increases in the intensity and frequency of precipitation events).

8.6.6.2 Erosion Control

8.6.6.2.1 Outer Dykes

The outer dykes should have a cover layer of at least 100 mm of fertile topsoil to promote establishment of an adequate vegetative cover wherever rip-rap is not utilized. Adequate vegetation should be established on dykes from the outside toe to 0.5 m below the top of the embankment as measured on the slope. Perennial-type, low-growing, spreading grasses that minimize erosion and can be mowed are most satisfactory for seeding on dykes. Additional erosion control may also be necessary on the exterior dyke slope to protect the embankment from erosion due surface water runoff and flooding of a watercourse (including climate change considerations).

8.6.6.2.2 Inner Dykes

Alternate erosion control on the interior dyke slopes has become necessary for ponds because of problems associated with mowing equipment not designed to run on slopes as well as a lack of maintenance by the System Owner. The inner dykes should have a cover of at least 200 mm of pit run gravel or other material graded in a manner to discourage the establishment of any vegetation. The material should be spread on dykes from the inside toe to the top of the embankment. Clean and sound rip-rap or an acceptable equal should be placed from 0.3 m above the high-water mark to 0.6 m below the low water mark (measured on the vertical). Maximum size of rock used should not exceed 150 mm.

8.6.6.2.3 Top of Embankment

The top of the embankment used for access around the perimeter of the dykes should have a cover layer of at least 300 mm similar to the one described in Section 8.6.6.

8.6.6.2.4 Additional Erosion Protection

Rip-rap or some other acceptable method of erosion control is required as a minimum around all piping entrances and exits. For aerated cells, the design should ensure erosion protection on the slopes and bottoms in the areas where turbulence will occur.

8.6.6.2.5 Erosion Control During Construction

Effective site erosion control should be provided during construction according to applicable provincial documents such as *Erosion and Sedimentation Control Handbook for Construction Sites* from Nova Scotia Environment, if available in the province of jurisdiction. An approved erosion control plan is required before construction begins.

8.6.6.3 Vegetation Control

A method should be specified which will prevent vegetation growth over the surface of the inner slope and top of the embankment.

8.6.6.4 Pond Bottom

8.6.6.4.1 Vertical Separation

For separation distance between the cell bottom and bedrock, refer to Section 8.6.6.4. Cell bottoms should be located sufficiently high above the groundwater level, in order to prevent inflow and/or liner damage. Potential change to groundwater elevations resulting from the impacts of climate change are to be considered.

8.6.6.5 Uniformity

The pond bottom should be as level as possible at all points. Finished elevations should not be more than 75 mm from the average elevation of the bottom.

8.6.6.5.1 Vegetation

The bottom should be cleared of vegetation and debris. Organic material thus removed should not be used in the dyke core construction, however, suitable topsoil relatively free of debris may be used as cover material on the outer slopes of the embankment as described in Section 8.6.6.

8.6.6.5.2 Permeability Tests

Permeability tests should be carried out on the soil material at each proposed stabilization basin site except in cases where the soil is unmistakably impervious. The permeability tests may take either of two forms:

- Laboratory tests on samples from below the proposed bottom of the stabilization basin and from the material to be used in the dykes.
- Field seepage tests can be conducted using either the Guelph Permeameter Method or the Pask Permeameter Method at a frequency of at least one test for every 2.0 ha of stabilization basin.

8.6.6.5.3 Interpretation of Hydraulic Conductivity Measurements

There can be major differences between laboratory and field hydraulic conductivity measurements. These differences are likely to occur because of complex geological and hydrogeological conditions, in-situ and errors in measurement methods. The ratio of K (in-situ) to K (laboratory) may be in the range of 0.38 to 64. The major reasons for higher field values are:

- Laboratory tests are generally run on homogeneous, clayey samples.
- Sand seams, fissures and other macrostructures in the field are not present in laboratory samples.
- Measurement of vertical K in the laboratory and horizontal K in the field.
- Changes in soil structure, chemical characteristics of the permeant, air entrapment in laboratory samples, and other errors associated with laboratory tests.

The value of K from a field test as described above may be obtained from the following equation:

$$K = (A/FDt) \times [In(h_1/h_2)]$$

(from *Measurement of the Hydraulic Conductivity of Fine-Grained Soils* by Olson and Daniel and published by the ANSI).

Where:

A	=	Area of standpipe.
T	=	Time for head change from h_1 to h_2.
D	=	Diameter of hole.
h	=	Head water above water table.
F	=	2.0 for a borehole with a flat bottom at an upper impervious boundary.
	or	
	=	2.75 for a cased borehole with a flat bottom in the middle of a deep soil layer.

Alternatively, if the laboratory tests show a permeability greater than 1 x 10^{-6} cm per second, or if the drop in head of the field test exceeds 10 mm per 24 hours, then provision should be made to make the soil more impermeable, as indicated in Section 8.6.6.5.6.

8.6.6.5.4 Soil

Soil used in constructing the pond bottom (not including liner) and dyke cores should be relatively incompressible and tight and compacted at or up to 4% above the optimum water content to at least 90% Standard Proctor Density. Soft pockets that would prevent sufficient compaction of the liner must be sub-excavated and replaced with suitable, compacted fill.

8.6.6.5.5 Liner

Stabilization ponds should be sealed such that seepage loss through the seal is as low as practicably possible. Liners consisting of soils or bentonite as well as synthetic liners may be considered, provided the permeability, durability and integrity of the proposed material can be satisfactorily demonstrated for anticipated conditions. Results of a testing program which substantiates the adequacy of the proposed liner must be incorporated into and/or accompany the Pre-design Report. Standard ASTM procedures or acceptable similar methods should be used for all tests. Where clay liners are used, precautions should be taken to avoid erosion and desiccation cracking prior to placing the system in operation.

8.6.6.5.6 Seepage Control Criterion for Clay Liners

The seepage control criterion for municipal wastewater stabilization ponds and aerated stabilization basins utilizing clay liners specifies a maximum hydraulic conductivity (K) for the pond liner as a function of the liner thickness (L) and water depth (D) by the equation:

$$Maximum\ K\ (m/s) = \frac{4.6 \times 10^{-8}\ m/s \times L(m)}{D\ (m) + L\ (m)}$$

(Where all units are in metres and seconds).

For example, a compacted clay liner that is 0.5 m thick must have a hydraulic conductivity of about 1.3×10^{-8} m/s (1.3×10^{-6} cm/s) or less. The "K" obtained by the above expression corresponds to a percolation rate of pond water of less than 40 m^3/day/ha at a water depth of 1.2 m.

8.6.6.6 Seepage Control Criterion for Synthetic Liners

For synthetic liners, seepage loss through the liner should not exceed the quantity equivalent through an adequate soil liner. For liner durability the minimum liner thickness for a HDPE liner should be 1.5 mm. The liner should be underlain by a sand layer with a minimum thickness of 150 mm.

8.6.6.7 Site Drainage

Surface drainage must be routed around and away from cells. Field tiles within the area enclosed by the berms must be located and blocked so as to prevent cell content leakage. Measures must be taken, where necessary, to avoid disruption of field tile and surface drainage of adjacent lands, by constructing drainage works to carry water around the site.

8.6.7 Design & Construction Procedures for Clay Liners

8.6.7.1 Delineation of Borrow Deposit

The first step in designing a compacted clay liner is delineating a relatively uniform deposit of suitable borrow material, preferably from the pond cut or from a nearby borrow area. The required volume of clayey soil is equal to the surface area of the pond interior times the liner thickness (measured perpendicular to the bottom and side slope surfaces). A large reserve volume is recommended to ensure that there is indeed sufficient clay volume after removing silt and sand pockets and other unsuitable materials.

8.6.7.2 Liner Thickness

Recommended minimum compacted clay liner thicknesses are 0.5 m on the pond bottom and 0.7 m on the side slopes (to allow for weathering, variations in actual thickness, pockets of poor-quality material that escape detection, etc.). If a clay core in the dyke is preferred over an upstream clay blanket liner, then the core should be well keyed into the bottom liner. A minimum core width of 3.0 m is suggested to allow economic and proper placement and compaction of the clay using large earth-moving equipment.

8.6.7.3 Hydraulic Conductivity of Compacted Clay

The in situ hydraulic conductivity of the compacted clay liner should be predicted from laboratory tests on the proposed clay borrow material. Several samples should be selected representing the range of material within the designated borrow zone, not just the better material. Permeability tests should be performed on the samples compacted to the required density (i.e., 95% of standard Proctor maximum dry density) at a moisture content anticipated in the field. It is recommended that the sensitivity of the compacted clay hydraulic conductivity to variations in density and moisture content be determined. The designer must be prepared to ensure that the soil is brought to the specified moisture content (i.e., by wetting), unless the natural moisture content is already suitable.

A laboratory value for K should be calculated from the weighted average of the individual tests. The weighting of each test value should be according to the estimated percent of the borrow volume that the individual sample represents.

It is recommended that the liner design be based on a K in situ that is one order of magnitude larger than the average K (lab):

$$K\ (design) = K(in\ situ) = 10 \times average\ K\ (lab)$$

The increase in the K value is a factor of safety to allow for the effects of macro-structure, poor quality borrow, etc., in the field. The K (design) and liner thickness values should meet the seepage criteria outlined in Section 8.6.6.5.6. If K (design) is too high, the more selective borrowing or adjustment of compaction moisture content could be investigated. Otherwise, an alternative liner material will be required.

8.6.7.4 Subgrade Preparation

Clay should not be placed directly over gravel or other materials that do not provide an adequate filter to prevent piping erosion of the liner.

8.6.7.5 Liner Material Placement & Compaction

The clay should be placed in uniform, horizontal lifts of about 150 mm maximum loose thickness. The liner should be constructed in at least three lifts. Thin lifts ensure more uniform density, better bonding between lifts and reduces the likelihood of continuous seepage channels existing in the liner. Large lumps, cobbles and other undesirable materials are more easily identified in thin lifts. Lumps of soil greater than 100 mm in maximum dimension should be broken up prior to compaction. As far as practical, the liner should be built up in a uniform fashion over the pond area, in order to avoid sections of butted fill where seepage paths may develop.

Each lift should be compacted within the specified moisture content range to the required density using heavy, self-propelled sheepsfoot compactors. Lift surfaces that have been allowed to dry out should be scarified prior to placing of the next lift. Lift surfaces that have degraded due to precipitation or other factors should be either removed or allowed to dry to the required moisture content and then be re-compacted.

The completed liner should be smoothed out with a smooth-barrel compactor to reduce the liner surface area exposed to water absorption and swelling. The liner base should not be allowed to dry out or be exposed to freezing temperatures. Ideally, the liner should be flooded as soon as possible after construction and acceptance.

8.6.7.6 Construction Control

The most important form of quality control during construction of compacted clay liners will be observation and direction by the engineer. The characteristics of the desired liner material should be established in as much detail as possible (i.e., by colour, texture, moisture content, plasticity, or characteristic features such as the mineralogy of pebbles in till). Quick visual or index test identification by experienced field personnel is probably the best way to detect poor quality material. An indirect but simple way of controlling liner quality is to perform frequent in situ density and moisture content tests. The density and moisture content may then be related to hydraulic conductivity by the relationships established during the laboratory test program (see Section 8.6.7.3). The frequency of tests should be increased when soil conditions are variable. The tests may be used to statistically evaluate the overall liner properties and to assess suspect zones in the liner.

In situ density and moisture content tests should be carried out on a routine basis for each lift. Tests should be conducted on a grid pattern (e.g., approximately 30 x 30 m to 60 x 60 m grids for large ponds and at closer spacing for small ponds) and in suspect areas.

The completed liner may be assessed by performing in situ infiltration tests, which may be theoretically related to hydraulic conductivity values (see Section 8.6.7.3). It should be noted that the compacted clay liner is most likely to be partially saturated at the end of construction. The presence of 5 to 10% air voids will result in an unsaturated K value that is somewhat higher than the saturated K value.

The completed liner may also be cored, and the hydraulic conductivity of a trimmed sample can be tested in a suitable permeameter (i.e., odometer falling head tests or triaxial constant head tests). All holes created in the liner due to tests, stakes, or other circumstances should be backfilled with well-compacted liner material.

8.6.7.7 Planning

The most important aspect of constructing a compacted clay liner may be the planning stage when the inspection engineer’s role is defined, contract specifications are prepared, and construction strategies are worked out. The engineer must have an adequate degree of control over material selection and methods of placement. The work procedure must be flexible with respect to earth movement.

Ideally, the borrow for a compacted clay liner would be the cut material just below the eventual pond invert. Thus, material may be cut and placed in a single operation for much of the pond liner area, although some stockpiling of borrow may be inevitable.

The lower lift of the liner might consist of reworked native soil broken up by tilling and re-compacted to eliminate fissures, etc. Nevertheless, the contract should allow for selective borrowing of cut material for liner use, stockpiling, removal of undesirable materials, and possible additional borrowing outside of the cut area.

8.6.8 Prefilling

Prefilling the pond should be considered in order to protect the liner, prevent weed growth, reduce odour, and maintain moisture content of the seal, however, the dykes must be completely prepared as described in Section 8.6.6.2 before the introduction of water.

8.6.9 Influent Lines

8.6.9.1 Material

Any generally accepted material for underground sewer construction will be given consideration for the influent line to the pond. Unlined corrugated metal pipe should be avoided, however, due to corrosion problems. In material selection, consideration must be given to the quality of the wastes, exceptionally heavy external loadings, abrasion, soft foundations, and similar problems.

8.6.9.2 Surcharging

The design and construction of influent piping should ensure that where surcharging exists, due to the head of the pond, no adverse effects will result. These effects should include basement flooding and overtopping of manholes.

8.6.9.3 Force Mains

Force mains terminating in a wastewater stabilization basin should be fitted with an isolation valve immediately upstream of the stabilization basin.

8.6.9.4 Location

Influent lines should be located along the bottom of the pond so that the top of the pipe is below the average elevation of the pond seal, however, the pipe should have adequate seal below it. The use of an exposed dyke to carry the influent line to the discharge points is prohibited.

8.6.9.5 Point of Discharge

The influent line to a square single celled pond should be essentially centre discharging. Each square cell of a multiple celled pond operated in parallel should have its own near centre inlet, but this does not apply to those cells following the primary cell when series operation alone is used. Influent lines to single celled rectangular ponds should terminate at approximately the third point farthest from the outlet structure. Influent and effluent piping should be located to minimize short-circuiting within the pond. Consideration should be given to multi-influent discharge points for primary cells of 5 ha or larger.

All aerated cells should have influent lines which distribute the load within the mixing zone of the aeration equipment. Consideration of multiple inlets should be closely evaluated for any diffused aeration system. For aerated stabilization basins the inlet pipe may go directly through the dyke and end at the toe of the inner slope.

8.6.9.6 Influent Discharge Apron

Inlet pipes should terminate 450 mm above the cell bottom.

The end of the discharge line should rest on a suitable concrete apron large enough to prevent the terminal influent velocity at the end of the apron from causing soil erosion. A minimum size apron of 1 m^2 should be provided.

8.6.9.7 Pipe Size

The influent system should be sized to permit peak raw wastewater flow to be directed to any one of the primary cells. Influent piping should provide a minimum scouring velocity of 0.6 m/s.

8.6.10 Control Structure & Interconnecting Piping

8.6.10.1 Structure

Plant design should consider the use of multi-purpose control structures to facilitate normal operational functions such as drawdown and flow distribution, flow and depth measurement, sampling, pumps for re-circulation, chemical additions and mixing and minimization of the number of construction sites within the dykes.

Control structures should:

- Be accessible for maintenance and adjustment of controls.
- Be adequately ventilated for safety and to minimize corrosion.
- Be locked to discourage vandalism.
- Contain controls to permit water level and flowrate control, complete shut-off, and complete draining.

- Be constructed of non-corrodible materials (metal-on-metal contact in controls should be of similar alloys to discourage electro-chemical reactions).
- Be located to minimize short-circuiting within the cell and avoid freezing and ice damage.

Recommended devices to regulate water level are valves, slide tubes, dual slide gates, or effluent chambers complete with a water level regulating weir. Regulators should be designed so that they can be pre-set to stop flows at any pond elevation.

8.6.10.2 Piping

All piping should be of DI, PVC, or HDPE. The piping should not be located within or below the liner. Pipes should be anchored with adequate erosion control.

8.6.10.2.1 Drawdown Structure Piping

Submerged Takeoffs

For ponds designed for shallow or variable depth operations, submerged takeoffs are recommended. Intakes should be located a minimum of 3.0 m from the toe of the dyke and 0.6 m from the top of the liner and should employ vertical withdrawal.

Multi-level Takeoffs

For ponds that are designed deep enough to permit stratification of pond content, multiple takeoffs are recommended. There should be a minimum of three withdrawal pipes at different elevations. The bottom pipe should conform to a submerged takeoff. The others should utilize horizontal entrance. Adequate structural support should be provided.

Surface Takeoffs

For use under constant discharge conditions and/or relatively shallow ponds under warm weather conditions, surface overflow-type withdrawal is recommended. Design should evaluate floating weir box or slide tube entrance with baffles for scum control.

Maintenance Drawdown

All ponds should have a pond drain to allow complete emptying, either by gravity or pumping, for maintenance. These should be incorporated into the above-described structures.

In aerated stabilization basins where a diffused air aeration system and submerged air headers are used, provision should be made to drain each stabilization basin (independently of others) below the level of the air header.

Emergency Overflow

All cells should be provided with an emergency overflow system which overflows when the liquid reaches within 0.6 m of the top of the berms.

8.6.10.2.2 Hydraulic Capacity

The hydraulic capacity for continuous discharge structures and piping should allow for at least the expected future peak wastewater pumping rate.

The hydraulic capacity for controlled discharge systems should permit transfer of water at a rate of 150 mm of pond water depth per day at the available head.

8.6.10.2.3 Interconnecting Piping

Interconnecting piping for multiple unit installations operated in series should be valved or provided with other arrangements to regulate flow between structures and permit flexible depth control. The interconnecting pipe to the secondary cell should discharge horizontally near the stabilization basin bottom to minimize need for erosion control measures and should be located as near the dividing dyke as construction permits. Interconnection piping should enable parallel or series flow patterns between cells.

8.6.10.3 Location

The outlet structure and the inter-connecting pipes should be located:

- Away from the corners where floating solids accumulate.
- On the windward side to prevent short-circuiting.

8.6.11 Miscellaneous

8.6.11.1 Groundwater Monitoring

An approved system of wells or lysimeters may be required around the perimeter of the pond site to facilitate groundwater monitoring. The need for such monitoring will be determined on a case-by-case basis.

8.6.11.2 Pond Level Gauges

Pond level gauges should be provided.

8.6.11.3 Service Building

A service building for laboratory and maintenance equipment should be provided, if required.

8.6.11.4 Liquid Depth Operation

Optimum liquid depth is influenced to some extent by stabilization basin area since circulation in larger installations permits greater liquid depth. The basic plan of operation may also influence depth. Facilities to permit operation at selected depths between 0.6 to 1.5 m are recommended for operational flexibility. Where winter operation is desirable, the operating level can be lowered before ice formation and gradually increased to 1.5 m by the retention of winter flows. In the spring, the level can be lowered to any desired depth at the time surface runoff and dilution water are generally at a maximum. Shallow operation can be maintained during the spring with gradual increased depths to discourage emergent vegetation in the summer months. In the fall, the levels can be lowered and again be ready for retention of winter storage.

8.6.11.5 Pre-Treatment & Post-Treatment

The wastewater should be treated by bar screens before entering the stabilization basin. The treated effluent should be disinfected prior to discharging into the receiving water.

8.7 Other Biological Systems

New biological treatment schemes with promising applicability in wastewater treatment may be considered if the required engineering data for new process evaluation is provided in accordance with Section 5.4.2. Several new biological systems are described below. These systems typically are manufactured by companies who hold proprietary designs and as proprietary information cannot be included in these Guidelines the design data presented is general in nature. A description of these systems mainly describing their application and typical loading rates is provided here.

8.7.1 Biological Aerated Filters

Biological Aerated Filters (BAFs) are submerged, granular media upflow filters, which treat wastewater by biologically converting carbonaceous and nitrogenous matter using biomass fixed to the media and physically capturing SS within the media. The filters are aerated to remove carbonaceous matter and convert ammonia-nitrogen to nitrates via nitrification. Non-aerated filters in the presence of supplemental carbonaceous organic matter can convert nitrates to nitrogen gas through denitrification.

Biological aerated filters are designed either as co-current backwash or countercurrent backwash systems. The co-current backwash design has a nozzle deck supporting a granular media that has a specific gravity greater than 1.0. Pre-treated wastewater is introduced under the nozzle deck and flows up through a slightly expanded media bed, and effluent leaves the filter from above the media. Process air is introduced just above the nozzle deck (the bed is not aerated for denitrification). During backwash, wash water and air scour are introduced below the nozzle deck and flow up through the bed. Wash water is pumped to the head of the plant or directly to solids handling.

The countercurrent backwash BAF operates under the same general principles, except that the granular media has a specific gravity less than 1.0, therefore, the media float and are retained from above by the nozzle deck. During backwash, wash water flows by gravity through the media. Process air is introduced below the media, therefore, scour air moves countercurrent to the wash water flow.

8.7.1.1 Design Features

The granular media bed for both designs typically is 3.0 to 4.0 m deep and the media are 3.0 to 6.0 mm in diameter. The media-specific surface area ranges from 500 to 2,000 m^2/m^3. The Contact Time (CT) in the media typically is 0.5 to 1.0 hour. The media bed is backwashed every 24 to 48 hours for 20 to 40 minutes using a wash water volume about three times the media volume. Backwash water from a single event is collected in a storage tank and returned to the head of the plant or directly to solids processing over a 1- to 2-hour period. Backwash water typically contains from 400 to 1,200 mg/L of SS. The backwash water recycle flow can represent up to 20% of the raw influent wastewater flow. Most manufacturers have estimated that solids production from the BAF system is comparable to that of a conventional activated sludge system. Effluent pollutant concentrations from a single BAF cell increases for approximately 30 minutes following a backwash event, so a minimum of four cells should be included in any design to dampen these spikes.

The nozzle deck features polyethylene nozzles that prevent media loss and assist In evenly distributing flow across the bed. The reported media loss from the BAF system is less than 2% per year. The nozzle openings are slightly smaller than the media and require that influent be pre-treated with a fine screen to prevent plugging. Headloss across the media bed can be more than 2.0 m prior to backwash. In existing installations, the filters are constructed above grade. The combination of the tall structure (6.0 m) and headloss across the bed requires pumping influent flow to the BAF in most situations. In addition, the co-current designs require pumping of wash water, which is significant, by intermittent, energy demand.

Process air is required in BAF cells that are removing carbonaceous organic matter (BOD_5) and are nitrifying ammonia-nitrogen. The process aeration system consists of coarse- to medium-bubble diffusers on a stainless-steel piping grid. As it is often difficult to access the aeration grid, the diffusers are constructed as simply and reliably as possible. The amount of air that must be added to the system is determined by the oxygen demand of the biomass. Energy for process air can represent more than 80% of the energy demand in a BAF system.

8.7.1.2 Configurations

Biological aerated filters can operate in different process configurations, depending on the plant, effluent goals, and wastewater characteristics. The process can follow either chemically assisted primary sedimentation or an activated sludge system. This level of treatment is required because of a BAF system's sensitivity to high influent BOD_5 and SS loadings. Following primary sedimentation, BAF cells can be operated for carbonaceous BOD_5 removal or, under lower loading rates (less than 1.5 kg $BOD_5/m^3 \cdot d$), for both carbonaceous BOD_5 and ammonia-nitrogen removal. A cell can operate in a nitrification mode following an activated sludge system or another BAF cell removing carbonaceous BOD_5. A denitrification BAF process can follow either an activated sludge or BAF system that is nitrifying.

8.7.1.3 Performance

The performance of BAFs in terms of allowable loading rates and effluent quality depends on influent wastewater quality and temperature. In general, higher organic or SS influent loadings result in higher effluent concentrations. Adequate water velocity is necessary to provide scouring of the biomass and even flow distribution across the media bed. Inadequate water velocity can result in premature bed plugging; this is especially true for denitrification reactors in which the effects of air scouring are not present.

Factors that positively affect complete nitrification include:

- Warm water temperature.
- Adequate aeration and good air distribution.
- Low carbonaceous BOD_5 and SS loading.

Denitrification usually requires methanol addition, and water velocities must be greater than 10 m/hour.

8.7.2 Moving Bed Biofilm Reactors

The patented Moving Bed Biofilm Reactor (MBBR) process was developed by Krüger Kaldnes. The basic concept of the MBBR is to have continuously operating, non-clogable biofilm reactors with no need for backwashing or return sludge flows, low head-loss, and high specific biofilm surface area. This is achieved by having the biomass grow on small carrier elements that move along with the water in the reactor. The movement is normally caused by coarse-bubble aeration in the aeration zone and mechanical mixing in an anoxic/anaerobic zone.

For small plants, mechanical mixers are omitted for simplicity reasons and pulse aeration for a few seconds a few times per day can be used to move the biofilm carriers in anoxic reactors.

The biofilm carrier elements are made of 0.96 specific gravity polyethylene and shaped like small cylinders, with a cross in the inside of the cylinder and longitudinal fins on the outside. To keep the biofilm elements in the reactor, a screen of perforated plate is placed at the outlet of the reactor. Agitation constantly moves the carrier elements over the surface of the screen; the scrubbing action prevents clogging.

Almost any size or shape tank can be retrofitted with the MBBR process. The filling of carrier elements in the reactor may be decided for each case, based on degree of treatment desired, organic, and hydraulic loading, temperature, and oxygen transfer capability. The reactor volume is totally mixed and consequently there is no "dea" space or unused space in the reactor. Organic loading rates for these reactors are typically in the order of 3.5 to 7.0 g BOD_5/m^2 of media surface area/d for BOD_5 removal and less than 3.5 g BOD_5/m^2 of media surface area/d for nitrification.

8.7.3 Membrane Bioreactors

Membrane Bioreactors consist of a suspended growth biological reactor (activated sludge system variation) integrated with a microfiltration membrane system. The key to the technology is the membrane separator which allows elevated levels of biomass to degrade or remove the soluble form of the organic pollutants from the waste stream. These systems typically operate in the nanofiltration or microfiltration range which results in removal of particles greater than 0.01 and 0.1 µm, respectively.

8.7.3.1 Configuration

Membrane bioreactors can be configured in several different ways, however, the two main configurations differ by those in which the membranes are submersed directly in the bioreactor and those which contain external membrane process tankage. When membrane modules are submersed into the bioreactor, they are in direct contact with the wastewater and sludge. A vacuum is created within the hollow fibres by the suction of a permeate pump. The treated water passes through the membrane, enters the hollow fibres, and is pumped out by the permeate pump. An air flow may be introduced to the bottom of the membrane module to create turbulence which scrubs and cleans the membrane fibres keeping them functioning at a high flux rate. The filtrate or permeate is then collected for reuse or discharge.

Outboard membrane processes operate in a similar manner, however, the membranes are contained in a separate tank through which the wastewater requiring filtration constantly flows. Again, air is often added for both treatment and membrane scouring purposes. The main difference between the two configurations lies in the membrane cleaning processes where membranes submersed within the aeration tanks must be removed for cleaning while outboard membranes are cleaned by evacuating the membrane tankage and providing for equalization during the cleaning procedures within the main aeration tank.

8.7.3.2 Process Description

The benefits of these processes are consistent effluent quality, reduced footprint, increased expansion capabilities within the same tankage, and ease of operation. Tertiary quality effluent is the normal output of a membrane bioreactor.

Virtually no solids are lost via the permeate stream and the wasting of solids is reduced. As a result, the sludge age can be very accurately determined. Nitrification for ammonia removal is easily achieved by optimizing reactor and sludge age to specific wastewater characteristics and effluent requirements. Absolute control of the nitrifiers results in high nitrification rates even in winter periods and under adverse and unstable conditions.

If required, denitrification can be achieved with membrane processes because, when operating at a MLSS of 15,000 mg/L and higher, the mixed liquor rapidly becomes anoxic in the absence of a continuous stream of air. Furthermore, the high level of biomass ensures that in the anoxic zone, at all times there is enough de-nitrifiers to efficiently convert the nitrates into nitrogen gas.

8.7.4 Recirculating Filters

Recirculating filters provide advanced secondary treatment of settled wastewater or septic tank effluent using sand, gravel, or other media. Recirculating filters consist of a lined excavation or structure, filled with uniform washed sand that is placed over an underdrain system. Through a distribution network the wastewater is dosed onto the surface and percolates through the media to the underdrain system. The underdrain system collects filter effluent and directs it to the recirculation tank for further processing or discharge.

8.7.4.1 Recirculating Sand Filters

Recirculating Sand Filters (RSFs) are aerobic, fixed-film bioreactors. Physical processes that occur in sand filters include straining and sedimentation which remove SS within the pores of the media. Chemical absorption of constituents such as phosphorus also occurs. Bioslimes from the growth of microorganisms develop as films on the sand particle surfaces. As the wastewater percolates through the sand the microorganisms in the slimes absorb the soluble and colloidal waste materials. The absorbed materials are either incorporated into a new cell mass or degraded under aerobic conditions to carbon dioxide and water.

8.7.4.2 Applications

Recirculating sand filters can be used for applications including single-family residences, large commercial establishments, and small communities. They can be used to pre-treat wastewater prior to subsurface infiltration and to meet water quality requirements before direct discharge to surface water. Recirculating sand filters are primarily used to treat domestic wastewater, but they have also been used successfully in treating wastewaters from restaurants and supermarkets, which are high in organic materials. recirculating filters can be used for both large and small flows and are frequently used where nitrogen removal is necessary.

8.7.4.3 System Components

Basic components of recirculating filters include a recirculation/dosing tank, pump and controls, distribution network, filter bed with an underdrain system, and a return line. The return line or the underdrain splits the flow to recycle a portion of the filtrate to the recirculation/dosing tank. A small volume of wastewater and filtrate is dosed to the filter surface on a timed cycle 1 to 3 times per hour. Recirculation ratios are typically between 3-to-1 and 5-to-1. The returned aerobic filtrate mixes with the anaerobic septic tank effluent in the recirculation tank before being reapplied to the filter.

There are many types of media used in packed-bed filters. The most common include washed, graded sand, however, pea gravel has generally replaced it in recent times. Other granular media which can be used include:

- Crushed glass.
- Garnet.
- Anthracite.
- Plastic.
- Expanded clay.
- Expanded shale.
- Open-cell foam.
- Extruded polystyrene.
- Bottom ash from coal-fired power plants.

Coarse-fibre synthetic textile materials are also used but are usually restricted to proprietary units.

Recirculation tanks consist of a tank, recirculation pump and controls, and a return filter water flow splitting device. Recirculation tanks store returned filtrate, mix the filtrate with the septic tank effluent, and store peak influent flows. The recirculation pump and controls are designed to dose a constant volume of mixed filtrate and septic tank effluent flow onto the filter on a timed cycle.

Distribution methods used include rigid pipe pressure networks with orifices or spray nozzles, and drip emitters. Rigid pipe pressure networks are the most commonly used method. Orifices with orifice shields, facing upward, minimize hole blockage by stones.

The most common flow splitting devices are ball float valves, proportional splitters, and stubbed sump dividers. The ball float valve is used where the recirculation tank is designed to remain full. The valve is connected to the return filtrate line inside the recirculation tank. The return line runs through the tank. The ball float valve is open when the water level is below the normally full level. When the tank fills from either the return filtrate or the influent flow, the ball float rises to close the valve, and the remaining filtrate is discharged from the system. The proportional splitters continuously divide the flow between return filtrate and the filtrate effluent. The stubbed sump splitter consists of a sump in which two pipes are stubbed into the bottom with their ends capped. In the crowns of each capped line, a series of equal-sized, pluggable holes are drilled. The return filtrate floods the sump, and the flow is split in proportion to the relative number of holes left open in each perforated capped pipe.

Most RSFs are constructed aboveground and with an open filter surface, however, in cold climates such as in Atlantic Canada, they should be placed in the ground to prevent freezing. The filter basin can be a lined excavation or fabricated tank. Typical liner materials are PVC and polypropylene. The system should be arranged to allow gravity drainage of lines to prevent freezing.

The underdrain system is located on the floor of the tank or lined excavation. The ends of the underdrains should be brought to the surface of the filter and with cleanouts. The underdrain outlet is cut in the basin wall such that the drain invert is at the floor elevation and the filter can be completely drained. The underdrain outlet invert elevation must be sufficiently above the recirculation tank inlet to accommodate a minimum of 0.1% slope on the return line and any elevation losses through the flow splitting device. The underdrain is covered with washed, durable gravel to provide a porous medium through which the filtrate can flow to the underdrain system.

8.7.4.4 System Variant

A variance on the recirculation sand filter is the Recirculating Textile Filter (RTF) which is a proprietary wastewater treatment system. Recirculating textile filters have proven to be a reliable, energy-efficient, and low-maintenance technology. Unlike other filter beds as described above, RTFs use a lightweight, compact, and easy-to-maintain textile fabric in the place of sand or stone. Typical arrangements allow for pumped liquor from the recirculation blend tank to be evenly distributed over a number of hanging lightweight, absorbent, engineered textile media. Bioslimes develop on the textile media in much the same way as on sand in RSFs. As the wastewater percolates between the textiles the microorganisms in the slimes absorb the soluble and colloidal waste materials. The absorbed materials are either incorporated into a new cell mass or degraded under aerobic conditions to carbon dioxide and water. Bioslimes will sloth off the textile and will return to the recirculation blend tank to mix with influent from the septic tank.

Additional components of the RTF system are similar to the RSF, septic tankage, recirculation blend tank, mixing valves, and pumping/distribution elements. The RTF can be housed in a FRP prefabricated tank. A large media area can be provided with increasing depths, which allow for smaller footprint requirements compared to RSFs. Recirculating textile filters may be modulated for phased development growth.

Consideration should be given to the layout of RTF units so as to provide the operator with good access to the media should it be required to be removed for maintenance/cleaning.

As with RSF pre-treatment is required. Pre-treatment devices should provide for effective removal of grit, debris and excessive oil or grease prior to the RTF units.

8.8 Miscellaneous

8.8.1 Fencing

Wastewater treatment plants, irrespective of their mode of treatment, should be enclosed with a suitable fence to preclude livestock and discourage trespassing. Fencing of minimum 1.8 m should be provided. The fence should be located with a minimal separation distance of 1.0 m from the top outside edge of the embankment/treatment tank/chamber. Fencing should not obstruct vehicle traffic on top of the dyke, or around process tanks/chambers. A vehicle access gate of sufficient width to accommodate equipment should be provided. All access gates should be provided with locks.

8.8.2 Access

An all-weather access road should be provided to the treatment plant site to allow year-round maintenance of the plant. Multiple access points provide resilience against restricted or blocked access resulting from extreme events (e.g., flooding, forest fires, and blow down events) which may be worsened by climate change. Review of the potential impacts on critical infrastructure will determine if a plant requires multiple access points.

8.8.3 Warning Signs

Appropriate signs should be provided along the fence around the plant to designate the nature of the plant and warn against trespassing. At least one sign should be provided on each side of the site and one for every 150 m of its perimeter.

8.8.4 Flow Measurement

Provisions for flow measurement should be provided on the outlet. Safe access to the device should be made to permit safe measurement. Flow meters should be equipped with data loggers for storage of historic flows. Preference is given to data loggers that do not require discrete software for interrogation by the operator.

8.8.5 Service Building

A service building for laboratory and maintenance equipment should be provided, if required.

8.9 References

Antoine, R.L., *Fixed Biological Surfaces - Wastewater Treatment - The Rotating Biological Contactor*, CRC Press 1976.

Feurstein, Donald L., *Predesign Surveys and Monitoring of Waste Disposal Systems*.

Ganczarczyk, Jerry J., *Activated Sludge Process - Theory and Practice*, Marcel Dekker, New York, 1983.

National Fire Protection Association. *20 Standard for the Installation of Stationary Pumps for Fire Protection*. NFPA, 2022.

National Fire Protection Association. *820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities*. NFPA, 2020.

Nova Scotia Department of Environment. Erosion and Sedimentation Control Handbook for Construction Sites. Province of Nova Scotia, 1998. <https://novascotia.ca/nse/surface.water/docs/ErosionSedimentControlHandbook.Construction.pdf>

Olson, R., Daniel, D. Measurement of the Hydraulic Conductivity of Fine-Grained Soils. ASTM Special Technical Publications, 1981.

Standards Council of Canada. *CSA W204:19 Flood Resilient Design of New Residential Communities*. Toronto: CSA Group, December 2019. <https://www.csagroup.org/store/product/CSA%20W204%3A19/>.

Tchobanoglous, George, Burton, Franklin L., Stensel, H. David, and Metcalf & Eddy. *Wastewater Engineering: Treatment and Reuse*. 4th Ed. / Revised by George Tchobanoglous, Franklin L. Burton, H. David Stensel. ed. McGraw-Hill Series in Civil and Environmental Engineering. Boston; Montreal: McGraw-Hill, 2003.

U.S. Environmental Protection Agency, *Aerobic Biological Wastewater Treatment Facilities*, March 1977. https://nepis.epa.gov/Exe/ZyPDF.cgi/9100NX8X.PDF?Dockey=9100NX8X.PDF

U.S. Environmental Protection Agency, *An Emerging Technology – The Biological Aerated Filter*, July 1983. https://nepis.epa.gov/Exe/ZyPDF.cgi/200045HF.PDF?Dockey=200045HF.PDF

U.S. Environmental Protection Agency, *Design Information on Rotating Biological Contactors*, June 1984. https://nepis.epa.gov/Exe/ZyPDF.cgi/20007IWK.PDF?Dockey=20007IWK.PDF

U.S. Environmental Protection Agency, *Environmental Regulations and Technology - Autothermal Thermophilic Aerobic Digestion of Municipal Wastewater Sludge*, September 1990. https://nepis.epa.gov/Exe/ZyPDF.cgi/300043GC.PDF?Dockey=300043GC.PDF

U.S. Environmental Protection Agency, *Water Engineering and Management, Rotating Biological Contactors*, June 1984, https://nepis.epa.gov/Exe/ZyPDF.cgi/20007IWK.PDF?Dockey=20007IWK.PDF

U.S. Environmental Protection Agency. Clean Water Act Analytical Methods. USEPA, April 2022. < https://www.epa.gov/cwa-methods>

Vesilind, P. *Wastewater Treatment Plant Design*. Water Environment Federation (WEF), 2003.

Water Environment Federation, *Membrane Systems for Wastewater Treatment*, 2006.

Water Pollution Control Federation, *Manual of Practice FD-3 - Pretreatment of Industrial Wastes*, 1981.

Water Pollution Control Federation, *Manual of Practice FD-9 - Sludge Stabilization*, 1985.

Water Pollution Control Federation, Manual of Practice OM-9 - Activated Sludge, 1987.

Table of Contents

Chapter 9 Treated Effluent Disinfection

9.1 Basis for Disinfection of Treated Effluent

Disinfection of WWTP effluent should be required in all cases, unless confirmed otherwise by the Regulatory Authority. The Regulatory Authority should be consulted for province-specific requirements when designing a disinfection system.

The design should consider meeting both the bacterial standards and the disinfectant residual limit in the effluent. The disinfection process should be selected after due consideration of:

- Waste characteristics.
- Type of treatment process provided prior to disinfection.
- Waste flowrates.
- pH of waste.
- Disinfectant demand rates.
- Current technology application.
- Cost of equipment and chemicals.
- Power cost.
- Maintenance requirements.

9.2 Forms of Disinfection

Ultraviolet radiation and chlorination are the most commonly used methods for wastewater effluent disinfection. The forms most often used for chlorination are chlorine and calcium or sodium hypochlorite. Other disinfectants, including chlorine dioxide, ozone, bromine, peracetic acid, or passive solar may be accepted by the Regulatory Authority in individual cases. Peracetic acid is an emerging technology that may be relevant for small WWTPs that have neither converted to UV nor installed dechlorination. If any form of chlorination is used, dechlorination should be used to meet the required effluent limits of 0.02 mg/L TRC for all systems. The use of chlorination tablets along with dechlorination tablets may be considered for small systems.

9.3 Chlorine Disinfection

9.3.1 Type

Chlorine is available for disinfection as chlorine gas, liquid sodium hypochlorite, and solid calcium hypochlorite tablets form. The type of chlorine should be carefully evaluated during the plant planning process. The use of chlorine gas or sodium hypochlorite will be most dependent on the size of the plant and the chlorine dose required. Large quantities of chlorine, such as are contained in ton cylinders and tank cars, can present a considerable hazard to plant personnel and to the surrounding area, should such containers develop leaks. The designer should consider the provisions of the latest edition of the *Transportation of Dangerous Goods Act* from Transport Canada, the *Environmental Protection Act* from the Government of Canada (specifically the *Environmental Emergency Regulations* from the Government of Canada), and the applicable provincial dangerous goods legislation when designing a disinfection system. Both monetary cost and the potential public exposure to chlorine should be considered when making the final determination.

Storage and handling procedures should conform to the guidelines of the latest edition of the Chlorine Basics prepared by The Chlorine Institute, Inc. and by the Regulatory Authority having jurisdiction.

9.3.2 Dosage

For disinfection, the capacity should be adequate to produce an effluent that will meet the applicable bacterial limits specified by the Regulatory Authority for that installation. Required disinfection capacity will vary, depending on the uses and points of application of the disinfection chemical. The chlorination system should be designed on a rational basis and calculations justifying the equipment sizing and number of units should be submitted for the whole operating range of flowrates for the type of control to be used. System design considerations should include the controlling wastewater flow meter (sensitivity and location), telemetering equipment and chlorination controls. The system should be capable of maintaining a total chlorine residual of at least 0.5 mg/L following the chlorine contact chamber, to provide disinfection to typical levels required by the Regulatory Authority. For normal domestic wastewater, the dosages in Table 9.1 may be used as a guide in sizing chlorination facilities.

Table 9.1: Typical Chlorine Dosage Requirements

Type of Treatment	Dosage
Trickling filter/RBC plant effluent	10 mg/L
Activated sludge/RTF plant effluent	8 mg/L
Tertiary filtration effluent	6 mg/L
Nitrified effluent	6 mg/L

9.3.3 Containers

9.3.3.1 Cylinders

Seventy (70) kg cylinders should be used when chlorine demand is less than 70 kg per day. Cylinders should be stored in an upright position with adequate support brackets and chains at 2/3-cylinder height for each cylinder. Refer to Section 5.8.

9.3.3.2 Ton Containers

The use of ton containers should be considered where the average daily chlorine consumption is over 70 kg. A hoist or crane with a capacity of at least 2,000 kg should be provided for the handling of the ton containers. Refer to Section 5.8.

9.3.3.3 Tank Cars

Refer to the latest edition of *Recommended Standards for Wastewater Facilities* from the Wastewater Committee of the Great Lakes - Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers.

9.3.3.4 Liquid Hypochlorite Solutions

Storage containers for hypochlorite solutions should be of sturdy, non-metallic lined construction and should be provided with secure tank tops and pressure relief and overflow piping. Storage tanks should be either located or vented outside. Provision should be made for adequate protection from light and extreme temperatures. Tanks should be located where leakage will not cause corrosion or damage to other equipment. A means of secondary containment should be provided to contain spills and facilitate cleanup. Due to deterioration of hypochlorite solutions over time, it is recommended that containers not be sized to hold more than 1 month's needs. At larger plants and locations where delivery is not a problem, it may be desirable to limit on-site storage to 1 week. Refer to Section 5.8.

9.3.3.5 Dry Hypochlorite Compounds

Dry hypochlorite compounds should be kept in tightly closed containers and stored in a cool, dry location. Some means of dust control should be considered, depending on the size of the plant and the quantity of compound used. Refer to Section 5.8.

9.3.4 Equipment

9.3.4.1 Scales

Scales of proper size should be provided at all plants using chlorine gas. At large plants, scales of the indicating and recording type are recommended. At the least a platform scale should be provided. Scales should be of corrosion-resistant material. Scales should be set on grade, or a ramp should be built to facilitate the moving of cylinders on and off the scale platform. Scales should be provided to accommodate the maximum number of containers online.

9.3.4.2 Evaporators

Where a manifold connecting several cylinders or 1-ton containers will be required to supply sufficient chlorine, consideration should be given to the installation of a chlorine evaporator to produce the quantity of gas required.

9.3.4.3 Chlorinators

All chlorinators should be vacuum-type chlorinators and should have automatic switchover capability.

9.3.4.4 Hypochlorite Metering

Application of hypochlorite for the purpose of disinfection should be by metering pumps specifically designed for this purpose. Calcium hypochlorite should be initially mixed in a make-up tank prior to any chlorination purpose.

9.3.4.5 Diffusers

A chlorine solution diffuser should be placed ahead of the contact tank and near the mixing area.

9.3.4.6 Mixing

The disinfectant should be positively mixed as rapidly as possible, with a complete mix being affected in three seconds. This may be accomplished by either the use of a turbulent flow regime or a mechanical flash mixer.

9.3.4.7 Contact Time & Tank

For a chlorination system, a minimum contact period of 15 minutes at design peak hourly flow or maximum rate of pumping should be provided after thorough mixing. For evaluation of existing chlorine contact tanks, field tracer studies should be done to assure adequate CT.

The chlorine contact tank should be constructed to reduce short-circuiting of flow to a practical minimum. Tanks not provided with continuous mixing should be provided with “over-and-under” or “end-around” baffling to minimize short-circuiting.

The tank should be designed to facilitate maintenance and cleaning without reducing effectiveness of disinfection. Duplicate tanks, mechanical scrapers, or portable deck-level vacuum cleaning equipment should be provided. Consideration should be given to providing skimming devices on all contact tanks. Covered tanks are discouraged.

9.3.4.8 Piping & Connections

Piping systems should be as simple as possible, specifically selected and manufactured to be suitable for chlorine service, with a minimum number of joints. Piping should be well supported and protected against temperature extremes.

Due to the corrosiveness of wet chlorine, all lines designated to handle dry chlorine should be protected from the entrance of water or air containing water. Even minute traces of water added to chlorine results in a corrosive attack. Low pressure lines made of hard rubber, saran-lined, rubber-lined, polyethylene, PVC, or other approved materials are satisfactory for wet chlorine or aqueous solutions of chlorine.

The chlorine system piping and valves should be color coded and labeled to distinguish it from other plant piping. Refer to Section 5.5.5. Where sulfur dioxide is used, the piping and fittings for chlorine and sulfur dioxide systems should be designed so that interconnection between the two systems cannot occur. Minimum 25 mm diameter piping should be used.

9.3.4.9 Standby Equipment & Spare Parts

Standby equipment of sufficient capacity should be available to replace the largest unit during shut-downs. Spare parts should be available for all disinfection equipment to replace parts which are subject to wear and breakage.

9.3.4.10 Chlorinator Water Supply

An ample supply of water should be available for operating the chlorinator. Where a booster pump is required, duplicate equipment should be provided, and, when necessary, standby power as well. Protection of a potable water supply should conform to the requirements of Section 4.2.9. Adequately filtered plant effluent should be considered for use in the chlorinator.

9.3.4.11 Leak Detection & Controls

A bottle of 56% ammonium hydroxide solution should be available for detecting chlorine leaks. Where 1-ton containers or tank cars are used, a leak repair kit approved by the Chlorine Institute should be provided. Consideration should be given to the provision of caustic soda solution reaction tanks for absorbing the contents of leaking 1-ton containers where such containers are in use. Automatic gas detection and related alarm equipment should be provided.

9.3.4.12 Cylinder & Container Handling

Chlorine cylinders should be conveyed by a wheeled cart.

Handling of 1-ton containers requires hoisting equipment. It is desirable to use a power-operated hoist and travel particularly when it is necessary to change containers frequently. All hoists must be rated for full load, including the weight of the empty containers and lifting tackle. Hoisting equipment under normal duty service must be visually inspected by the operator for damage before each use (minimum monthly) and have an annual record inspection performed.

9.3.4.13 Chlorinator Alarms

Each chlorinator in large plants should be equipped with a vacuum switch that should close or open a contact (and start an alarm) when there is an unusually high or low vacuum in the line from the chlorinator to the injector. Medium size plants should include such vacuum switch-alarm systems.

9.3.5 Housing Requirements

Under the *Environmental Emergency Regulations* from the Government of Canada, anyone storing or using a listed substance above the specified thresholds, or who has a container with a capacity for that substance in excess of the specified quantity, will have to notify Environment and Climate Change Canada of the place where the substance is held, along with the maximum expected quantity and the size of the largest container for that substance. If both the above criteria are exceeded, the System Owner is required to prepare and implement an environmental emergency plan and notify Environment and Climate Change Canada accordingly.

Chemical buildings or storage areas should be provided with adequate warning signs, conspicuously displayed where identifiable hazards exist, a storage area for SDS as set out under the *Hazardous Products Act* from the Government of Canada and associated *Controlled Products Regulations* from the Government of Canada. All storage containers should be conspicuously labelled with a WHMIS label that includes:

- The product name.
- The supplier name.
- Hazard symbol(s).
- Risk.
- Precautionary measures.
- First aid measures.

9.3.5.1 Gas Chlorine Feed & Storage Rooms

If gas chlorination equipment and chlorine cylinders are to be in a building used for other purposes, a gas-tight partition should separate this room from any other portion of the building. Doors to the chlorinator room should open only to the outside of the building and should be equipped with panic hardware. Such rooms should be at ground level and should permit easy access to all equipment. The building should be of fireproof material. The distance from any point in the room and the outside door should not exceed 5 m.

The storage area should be separated from the feed area. A "DANGER" sign should be placed on the door and safety precaution instructions to start up and shut-down should be placed at a visible location on the wall. Full and empty chlorine cylinders should be stored separately and should be chained to the wall in the vertical position. Cylinders should not be stored near flammable materials, heating or ventilation units, elevator shafts and on uneven or subsurface floors.

Chlorination equipment should be situated as close to the application point as reasonably possible. For additional safety considerations, refer to Section 5.8.

9.3.5.2 Inspection Window

A clear glass, gas-tight window should be installed in an exterior door or interior wall of the chlorinator room to permit the chlorinator to be viewed without entering the room.

9.3.5.3 Drains

Floor drains are not permitted in chlorine gas feed and/or storage rooms, except in installations using evaporators. Where approved, floor drains must be constructed of corrosion-resistant materials and must discharge to a drainage system separate from the rest of the treatment plant.

9.3.5.4 Heat

The chlorinator room should be provided with a means of heating so that a temperature of at least 16°C can be maintained, but the room should be protected from excess heat. Cylinders should be kept at essentially room temperature. If liquid hypochlorite solution is used, the containers may be located in an unheated area.

9.3.5.5 Ventilation

With gas chlorination systems, forced, mechanical ventilation should be installed which will provide one complete air change per minute when the room is occupied. The entrance to the air exhaust duct from the room should be 300 mm above the floor and the point of discharge should be so located as not to contaminate the air inlet to any buildings or present a hazard to inhabited areas. The air inlet should be located near the ceiling on the opposite side of the room so as to provide cross ventilation with air and at such temperatures that will not adversely affect the chlorination equipment. The outside air inlet should be at least 0.9 m above grade. The vent hose from the chlorinator should discharge above grade to the atmosphere. Where public exposure may be extensive such as residential or densely populated areas, scrubbers may be required on ventilation discharge.

9.3.5.6 Vents

All chlorinators should have a pressure/vacuum relief vent system. Each chlorinator should have a dedicated vent which should be carried to the outside atmosphere, without traps, to a safe area. The ends of the vent lines should point down, be covered with a copper wire screen to exclude insects and should not be more than 7.5 m above the chlorinator. The line should have a slight downward pitch from the high point (directly above the chlorinator) to drain any condensate away from the chlorinator. It is acceptable to run the vent vertically (but no more than 7.5 m) above the chlorinator to the roof, with a 180° return bend at the exit.

Each external chlorine pressure-reducing valve should be checked to see if it is provided with a vent; some are not vented, depending on the chlorine capacity. When supplied, these vents should drain away from the valves. These valves should be located high enough so that the individual drains will have a continuous downgrade to the outside atmosphere.

Evaporators have a steam vapour vent which can be manifolded together and discharged to the atmosphere without traps.

9.3.5.7 Electrical Controls & Ambient Gas Detectors

Switches for fans and lights should be outside of the room at the entrance. A labeled signal light indicating fan operation should be provided at each entrance if the fan can be controlled from more than one point. The controls for the fans and lights should be such that they will automatically operate when the door is open. All electrical equipment should be vapour-proof. Fans and lights should be on the same on-and-off switch whenever possible. An ambient chlorine gas detector should be provided in the chlorine storage room. The gas detector should be interlocked with the fan and audible or visual alarms.

9.3.5.8 Respiratory Protection

A self-contained air-supply breathing apparatus in good operating condition, meeting the requirements of the latest edition of *CSA Z94.4 Selection, use and care of respirators*, should be available at all installations where chlorine gas is handled. This equipment should be stored outside of any room where chlorine is used or stored. Instructions for using, testing, and replacing parts and air tanks should be posted. The units should use compressed air, have at least 30-minute capacity and be compatible with the units used by the fire department responsible for the plant.

9.3.5.9 Safety Equipment

Safety equipment required includes a first-aid kit, a fire extinguisher, goggles and gloves, a chlorine container repair kit, and an emergency eyewash and shower. Refer to Section 5.8.

9.3.5.10 Hypochlorite Feed & Storage Rooms

Chemicals containing chlorine compound should be stored in a separate room used for that purpose only. The room should be of fire-resistant construction and at or above grade. As heat and light affect the shelf life of sodium hypochlorite, the storage area should be kept cool and be protected from direct sunlight.

Calcium hypochlorite should be kept dry and covered. The storage area must not be serviced by automatic sprinkler systems. When heated above 170°C, calcium hypochlorite releases oxygen. For this reason, calcium hypochlorite must be kept away from flammable materials. Calcium hypochlorite storage areas should be provided with an exhaust system for the purpose of dust removal.

9.3.6 Sampling & Control

9.3.6.1 Sampling

Facilities should be provided for sampling disinfected effluent after the contact chamber. Either grab or composite sampling, of the type and frequency required by the Regulatory Authority (and also by WSER if applicable) for the specific plant, should be made for effluent chlorine residual of the disinfected effluent. Automatic sampling should be considered for improved control where appropriate.

9.3.6.2 Testing & Control

Equipment should be provided for measuring chlorine residual, employing the standard Diethyl-p-phenylene diamine (DPD) test as a minimum. The equipment should enable residual measurement to the nearest 0.01 mg/L in the range below 0.5 mg/L. For control purposes, but not for regulatory compliance, the dechlorination chemical itself can be measured, where any measured amount of dechlorination chemical in the final effluent represents an absence of Total Residual Chlorine (TRC).

Demonstrated effective facilities for automatic chlorine residual analysis by amperometric titrator, recording, and proportioning systems should be installed where the discharge occurs at points requiring rigid bacteriological controls such as on public water supply watersheds, recreational watersheds, shellfish waters, or waters tributary thereto.

9.3.7 Methods of Dosage Control

An automated dosage control system should be used for all WWTPs. The controls should adjust the chlorine dosage rate within an appropriate lag time to accommodate fluctuations in effluent chlorine demand and chlorine residual due to changes in flow and wastewater effluent characteristics. Alarms and monitoring equipment are required to promptly alert the operator in the event of any malfunction, hazardous situation, or inadequately disinfected effluent associated with the chlorine supply, including metering equipment, leaks, or other problems. Consideration should be given to the sensitivity and public health importance of the receiving water, the level of operator oversight, and the risk of false analytical readings given the wastewater effluent characteristics when selecting the appropriate type of dosage control, from the types described below.

9.3.7.1 Open-Loop Flow-Proportional Control

This method varies the rate of chlorine feed in proportion to the wastewater flow signal from a metering device. The chlorine dosage rate is manually set, and the control device varies the rate in relation to the wastewater flowrate. The required chlorine dosage should be manually adjusted based on intermittently or automatically measured total chlorine residual at the end of the chlorine contact tank before dechlorination.

9.3.7.2 Closed-Loop Flow-Proportional Control

An online chlorine residual analyzer provides feedback to the chlorinator. The simplest method has one chlorine residual analyzer automatically collect and analyze a sample at the end of the chlorine contact tank but before dechlorination. The flow signal and dosage signal each separately control the added chlorine feed with a compound-loop arrangement (e.g., if the residual is above the pre-determined level and/or the chlorine feed rate is reduced).

A system with two chlorine analyzers can also be used, where one sample is automatically collected and analyzed at the end of the chlorine contact tank but before dechlorination, and a second sample is collected immediately downstream from the point of chlorination (diffuser). Both these dosage signals, as well as the flow signal, separately control the added chlorine feed with a compound-loop arrangement (e.g., if the residual is above the pre-determined level and/or the chlorine feed rate is reduced).

9.4 Dechlorination

9.4.1 General

Dechlorination is required for WWTPs that disinfect using any form chlorination, to an average of 0.02 mg/L TRC or less.

The most common dechlorination chemicals are sulphur compounds, particularly sulphur dioxide gas or aqueous solutions of sulphite or bisulphate. Tablet dechlorination systems are also available for small plants.

The type of dechlorination system should be carefully selected considering criteria including the following:

- Type of chemical storage required.
- Amount of chemical needed.
- Ease of operation.
- Compatibility with existing equipment.
- Safety.

9.4.2 Dosage

The dosage of dechlorination chemicals should depend on the residual chlorine in the effluent, the final residual chlorine limit, and the particular form of the dechlorinating chemical used. The most common dechlorinating agent is sulphite. The following forms of the compound are commonly used and yield sulphite (SO_2^-) when dissolved in water.

Table 9.2: Dechlorination Theoretical Dose

Dechlorination Chemical	Theoretical mg/L Required to Neutralize 1.0 mg/L Cl_2
Sodium thiosulfate (solution)	0.56
Sodium sulfite (tablet)	1.78
Sulfur dioxide (gas)	0.90
Sodium metabisulfite (solution)	1.34
Sodium bisulfite (solution)	1.46

Theoretical values may be used for initial approximations, to size feed equipment with the consideration that under good mixing conditions 10% excess dechlorinating chemical is required above theoretical values. Excess sulphur dioxide may consume oxygen at a maximum of 1.0 mg DO for every 4 mg SO_2.

The liquid solutions come in various strengths. The solutions may need to be further diluted to provide the proper dose of sulphite.

9.4.3 Containers

Depending on the chemical selected for dechlorination, the storage containers will vary from gas cylinders, liquid in 190 L drums or dry compounds. Dilution tanks and mixing tanks will be necessary when using dry compounds and may be necessary when using liquid compounds to deliver the proper dosage. Solution containers should be covered to prevent evaporation and spills.

9.4.4 Feed Equipment, Mixing, & Contact Requirements

9.4.4.1 Equipment

In general, the same type of feeding equipment used for chlorine gas may be used with minor modifications for sulphur dioxide gas, however, the manufacturer should be contacted for specific equipment recommendations. No equipment should be alternately used for the two gases. The common type of dechlorination feed equipment utilizing sulphur compounds include vacuum solution feed of sulphur dioxide gas and a positive displacement pump for aqueous solutions of sulphite or bisulphate.

The selection of the type of feed equipment utilizing sulphur compounds should include consideration of the operator safety and overall public safety relative to the WWTPs proximity to populated areas and the security of gas cylinder storage. The selection and design of sulphur dioxide feeding equipment should consider that the gas re-liquifies quite easily. Special precautions must be taken when using ton containers to prevent re-liquefaction.

Where necessary to meet the operating ranges, multiple units should be provided for adequate peak capacity and to provide a sufficiently low feed rate on turn down to avoid depletion of the DO concentrations in the receiving waters.

9.4.4.2 Mixing Requirements

The dechlorination reaction with free or combined chlorine will generally occur within 15 to 20 seconds. Mechanical mixers are required unless the mixing facility will provide the required hydraulic turbulence to assure thorough and complete mixing. The high solubility of SO_2 prevents it from escaping during turbulence.

9.4.4.3 Contact Time

A minimum of 30 seconds for mixing and CT should be provided at the design peak hourly flow or maximum pumping rate. A suitable sampling point should be provided downstream of the contact zone. Consideration should be given to a means of reaeration to assure maintenance of an acceptable DO concentration in the stream following sulfonation.

9.4.4.4 Standby Equipment & Spare Parts

The same requirements apply as for chlorination systems.

9.4.4.5 Sulphonator Water Supply

The same requirements apply as for chlorination systems.

9.4.5 Housing Requirements

9.4.5.1 Feed & Storage Rooms

The requirements for housing SO_2 gas equipment should follow the same guidelines as used for chlorine gas though it is not best practice to have SO_2 and chlorine has house in the same space.

When using solutions of the dechlorinating compounds, the solutions may be stored in a room that meets the safety and handling requirements set forth in Section 5.8. The mixing, storage, and solution delivery areas must be designed to contain or route solution spillage or leakage away from traffic areas to an appropriate containment unit.

9.4.5.2 Protective & Respiratory Gear

The self-contained air-supply breathing apparatus equipment is the same as for chlorine (see Section 5.8). Leak repair kits of the type used for chlorine gas that are equipped with gasket material suitable for service with sulphur dioxide gas may be used. For additional safety considerations, see Section 5.8.

9.4.6 Sampling & Control

9.4.6.1 Sampling

Facilities should be included for sampling the dechlorinated effluent for residual chlorine. Provisions should be made to monitor for DO concentration after sulphonation when required by the Regulatory Authority.

9.4.6.2 Testing Control

Provision should be made for manual or automatic control of sulphonator feed rates based on chlorine residual measurement or flow.

9.4.7 Activated Carbon

Granular activated carbon may be used to dechlorinate wastewater effluent. The dechlorination reaction is dependent on the chemical state of the free chlorine, chlorine concentration and flowrate, physical characteristics of the carbon, and the wastewater characteristics. Consult vendors for specific design details.

9.5 Ultraviolet Disinfection

Ultraviolet disinfection process design, operating data, and experience are developed, but design standards are not well established. Expected performance of the UV disinfection units for the full operating range of flowrates should be based upon experience at similar full-scale installations or thoroughly documented prototype testing with the particular wastewater. Critical parameters for UV disinfection units are dependent upon manufacturer's design, lamp selection, tube materials, ballasts, configuration, control systems, and associated appurtenances. Ultraviolet disinfection systems are proprietary and the designer should consult vendors for specific design details, such as lamp module design, cleaning systems, safety requirements and spare part needs. Spare parts and materials need to be kept on-site. For additional details on critical design and operational parameters and UV equipment refer to the latest edition of *Ultraviolet Disinfection Technology for Municipal Wastewater Treatment Plant Applications in Canada* from the Environment and Climate Change Canada.

9.5.1 Lamp Type

Ultraviolet disinfection lamps should be low pressure-low intensity, low pressure-high intensity, or medium pressure-high intensity.

9.5.2 Channel Design & Hydraulics

Open channel designs with modular UV disinfection units that can be removed from the flow are typically used. At least two banks in series should be provided in each channel for disinfection reliability and to ensure uninterrupted service during tube cleaning or other required maintenance. The hydraulic properties of the system should be designed to simulate plug flow conditions without short circuiting under the full operating flow range. In addition, water level control should be provided to achieve the necessary exposure. The height of the wastewater above the top row of UV lamps must be rigidly controlled by a flap gate or weir for all flowrates. Hydraulic capacity and disinfection capacity of the system should both be considered during design. Also refer to Sections 5.5.2 and 5.5.4. Closed chamber units will be reviewed on a case-by-case basis in accordance with Section 5.4.2.

9.5.3 Transmittance

Ultraviolet light's ability to penetrate wastewater is measured with a spectrophotometer using the same wavelength (254 nm) that is produced by germicidal lamps. This measurement is called the percent transmittance and it is a function of all the factors which absorb or reflect UV light. As the percent transmission gets lower (higher absorbance) the ability of the UV light to penetrate the wastewater and reach target organisms decreases. The system designer must obtain samples of the wastewater during the worst conditions or carefully attempt to calculate the minimum expected UV transmission by testing wastewater from plants which have a similar influent and treatment process. The designer must also strictly define the disinfection limits since they determine the magnitude of the UV dose required. Designers should be aware of practical limits for low Ultraviolet Transmittance (UVT) below which UV disinfection may not be feasible and alternate disinfection methods may need to be considered. Factors listed below can affect Transmittance and should be considered.

9.5.3.1 Suspended Solids

Suspended solids in wastewater absorb or reflect the UV light before it can penetrate the solids to kill any occluded organisms. Ultraviolet light can penetrate into SS with longer CTs and higher intensities, but there is still a limit to the ability to kill the microorganisms.

9.5.3.2 Wastewater Iron Content

Iron can affect the UV disinfection by absorbing UV light. Dissolved iron, iron precipitate on quartz sleeves, and adsorption of iron by SS, bacterial floc, and other organic compounds, all decrease UVT. In cases where the wastewater has an iron level of > 0.3 mg/L, consideration should be given to pre-treatment or an alternate disinfection system.

9.5.3.3 Hardness

Calcium and magnesium salts cause water hardness. Hard water will precipitate on any warm or hot surface. Since the optimum operating temperature of the low-pressure mercury lamp is 40°C, the surface of the protective quartz sleeve will be warm. It will create a molecular layer of warm water where calcium and magnesium salts can be precipitated. These precipitates will prevent some of the UV light from entering the wastewater.

Waters which approach or are above 300 mg/L of hardness may require pilot testing of a UV system. This is especially important if very low flows or no flow situations are expected because they allow the water to warm up around the quartz sleeves and produce excessive coating.

9.5.3.4 Industrial Discharges

Periodic influxes of industrial wastewater may contain UV absorbing organic compounds, iron, or hardness, any of which may affect UV performance. Industries discharging wastes that contain such materials may be required to pre-treat their wastewater. Low concentrations of dye may be too diluted to be detected without using a spectrophotometer. Dye can readily absorb UV light thereby preventing UV disinfection.

9.5.4 Dosage

The UV dosage should be based on the design peak hourly flow. A UV dosage not less than 24 (mW·s)/cm^2 should be used after adjustments for maximum tube fouling, lamp output reduction after 8,760 hours of operation, and other energy absorption losses.

9.5.5 Operations, Safety, & Alarm Systems

Operator safety (electrical hazards and exposure to UV radiation) and UV equipment cleaning should be considered.

9.5.5.1 Electrical

Ground fault interruption circuitry or other CSA or cUL approved electrical safety features should be provided.

9.5.5.2 Ultraviolet Radiation

Equipment should be provided with safety interlocks that shut-down the UV banks or modules if moved out of their position or the liquid level drops below the top row of lamps in a horizontal system or exposes the top portion of the UV lamps in a vertical system. The vertical system may include light shields that allow a small portion of the tops of the lamps to be exposed to air without being a hazard.

Whenever low-pressure UV lamps are to be handled, personnel should be equipped with face safety shields rated to absorb light with wavelengths ranging from 200 to 400 nm and all exposed skin should be covered. Safety shields for medium-pressure UV lamps should be rated to absorb light with wavelengths ranging from 100 to 900 nm and all exposed skin should be covered. An arc welder's mask should be used with medium-pressure UV lamps.

9.5.5.3 Cleaning Mechanisms

Cleaning of sleeves and surfaces in contact with effluent is required due to fouling by iron, calcium, aluminum, manganese, and other organic and inorganic matter in the wastewater effluent. This fouling reduces UV light transmission significantly. Cleaning of the UV equipment should be considered. Approaches include out-of-channel cleaning tanks, manual wiping, and acid recirculation systems and/or automatic wiper systems. The size of the system and the likely rate of fouling should be considered when selecting a cleaning approach.

9.5.6 Controls

A PLC should be provided. An uninterruptable power supply with electrical surge protection should be provided for each PLC to retain program memory (e.g., process control program, last known set-points, and measured process/equipment status) through a power loss. A hard-wired backup for manual override should be provided in addition to automatic process control. Both automatic and manual controls should allow independent operation of each UV disinfection unit.

An alarm system should be provided to separately indicate lamp failure, low UV intensity, and any other cause of UV disinfection unit failure.

9.6 Ozone Disinfection

Refer to the latest edition of *Recommended Standards for Wastewater Facilities* from the Wastewater Committee of the Great Lakes - Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers and the *Ontario Design Guidelines for Sewage Works* from the Ontario Ministry of the Environment.

9.7 Peracetic Acid Disinfection

Design standards, operating data, and experience for this process are not well established, therefore, design of these systems should be based upon experience at similar full-scale installations or thoroughly documented prototype testing with the particular wastewater at site-specific conditions. Refer to the latest edition of *Peracetic Acid Disinfection - Implementation Considerations for Water Resource Recovery Facilities* from the Water Environment Federation.

9.8 References

Associated Engineering, (B.C.) Ltd. "Review of Municipal Wastewater Effluent." *Chlorination/Dechlorination Principles, Technologies and Practices*. Environment Canada, November 2003. <https://publications.gc.ca/collections/collection_2019/eccc/En84-36-2006-eng.pdf>.

CSA Group. *CAN/CSA-Z94.4-18 Selection, use, and care of respirators*. CSA, 2018. <https://www.csagroup.org/store/product/CAN%25100CSA-Z94.4-18/>.

Environment Canada, *Review of Municipal Effluent Chlorination/Dechlorination Principles, Technologies and Practices*. July 2013. <https://www.canada.ca/en/environment-climate-change/services/wastewater/resource-documents/municipal-effluent-chlorination-principles-technologies-practices.html>.

Government of Canada Justice Laws. *Environmental Emergency Regulations*. Minister of Justice, 2019. <https://laws.justice.gc.ca/PDF/SOR-2019-51.pdf>.

Government of Canada Justice Laws. *Hazardous Products Act*. Minister of Justice, 2022. <https://laws-lois.justice.gc.ca/PDF/H-3.pdf>.

Government of Canada, Ontario Ministry of the Environment and The Federation of Canadian Municipalities. *Ultraviolet Disinfection Technology for Municipal Wastewater Treatment Plant Applications in Canada*. Windsor, October 2003. <https://www.canada.ca/en/environment-climate-change/services/wastewater/resource-documents/ultraviolet-disinfection-technology-treatment-plants.html>.

Government of Canada. *Canadian Environmental Protection Act*. 14 September S.C. 1999, c. 33. <https://laws-lois.justice.gc.ca/PDF/C-15.31.pdf>.

Government of Canada. *Controlled Products Regulations* SOR/88-66. Government of Canada, 2015. <https://www.laws-lois.justice.gc.ca/eng/regulations/SOR-88-66/page-1.html>.

Government of Saskatchewan, *Guidelines for Chlorine Gas Use in Water and Wastewater Treatment*, 2004. <http://www.saskh2o.ca/DWBinder/EPB265Guidelines_for_ChlorineGasUse_in_Water.pdf>.

Nova Scotia Department of Environment, *Onsite Sewage Disposal Systems Technical Guidelines*, April 2013. <https://www.novascotia.ca/nse/wastewater/sewagedisposalguide.asp>.

Ontario Ministry of the Environment. *Design Guidelines for Sewage Works. Government of Ontario*. Government of Ontario, 2019. <https://www.ontario.ca/document/design-guidelines-sewage-works-0>.

Peracetic Acid Disinfection - Implementation Considerations for Water Resource Recovery Facilities Task Force of the Water Environment Federation. "Peracetic Acid Disinfection." *Implementation Considerations for Water Resource Recovery Facilities*. Alexandria: Water Environment Federation, 2020. <https://www.wef.org/resources/publications/books/peracetic-acid-disinfection/>.

Stover, E.L., et al. *EPA/625/1-86/021 Design Manual Municipal Wastewater Disinfection*. Ed. K.E Longley, et al. Cincinnati: U.S. Environmental Protection Agency, October 1986. <https://www.wbdg.org/FFC/EPA/EPACRIT/epa625_1_86_021.pdf>.

The Chlorine Institute. "Chlorine Basics." *Pamphlet 1*. May 2014. <https://bookstore.chlorineinstitute.org/mm5/merchant.mvc?Session_ID=472c8d9ce6b70f1d8d42a2c073a1a0ef&>.

Transport Canada. *Transportation of Dangerous Goods Act*. Government of Canada, 1992. <https://tc.canada.ca/en/dangerous-goods/transportation-dangerous-goods-act>.

Wastewater Committee of the Great Lakes - Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers. "Recommended Standards for Wastewater Facilities." *Policies for the Design, Review, and Approval of Plans and Specifications for Wastewater Collection and Treatment Facilities*. Albany: Health Research, Inc., Health Education Services Division, 2014. <https://www.health.state.mn.us/communities/environment/water/docs/tenstates/tenstatestan2014.pdf>.

Water Environment Federation, *Manual of Practice FD – 10, Wastewater Disinfection*, 1996.

Water Environment Federation. *The Chlorination/Dechlorination Handbook*. Vol. 1. January 2002. <https://www.accesswater.org/?id=-10074481>.

Water Environment Federation. *Ultraviolet Disinfection for Wastewater*. *Low-Dose Application Guidance for Secondary and Tertiary Discharges*. n.d. <https://www.wef.org/resources/publications/books/disinfection-book/>.

Water Supply Committee of the Great Lakes - Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers. "Recommended Standards for Water Works." *Policies for the Review and Approval of Plans and Specifications for Public Water Supplies.* Albany: Health Research Inc., Health Education Services Division, 2012. <https://www.health.state.mn.us/communities/environment/water/docs/tenstates/waterrev2012.pdf>.

Table of Contents

Chapter 10 Nutrient Removal & Tertiary Treatment

10.1 Basis for Nutrient Removal & Tertiary Treatment

Nutrient removal and tertiary treatment refers to the advanced treatment that targets the removal of nitrogen and phosphorus-based nutrients. Nutrient removal and tertiary treatment is not required in most cases, unless required by the Regulatory Authority on a case-by-case basis as the result of an ERA.

A number of process variations are available for consideration and should be evaluated on a case-by-case basis. Choice of process variation will be influenced by proposed plant size, characterization of wastewater, treatment technology, effluent discharge requirements, O&M considerations, and costs. Advanced wastewater treatment requires close attention and competent supervision, including routine laboratory control. Design should provide for flexibility an operation in various modes, if feasible.

10.1.1 Forms of Nutrient Removal

There are several forms of nutrient removal including physical, biological, and chemical. Physical nutrient removal can be achieved through the use of high-rate effluent filtration systems, membranes, micro screening, and high-rate clarification. Biological nutrient removal can also be achieved through modifications to several biological treatment arrangements. Chemical nutrient removal can be achieved with the use of various chemicals including:

- Aluminum salts.
- Iron salts.
- Lime.
- Chlorination.
- Air stripping.

10.2 Phosphorus Removal

This Chapter provides an overview of the governing *Canada-wide Strategy for the Management of Municipal Wastewater Effluent Technical Supplement 3: Standard Method and Contracting Provisions for the Environmental Risk Assessment* from the Canadian Council of Ministers of the Environment. Refer to the latest edition when determining requirements for phosphorus removal.

10.2.1 General

10.2.1.1 Applicability

The following factors should be considered when determining the need for phosphorus control at WWTPs:

- The present and future phosphorus loadings from the existing municipal WWTP to the receiving water.
- The background phosphorus levels in the receiving water and the effects of these levels on the rate of eutrophication along the entire length of receiving waters.
- The predicted response of the receiving water to increased phosphorus loadings.
- The existing and desired water quality of the receiving water along its entire length.

- The existing and projected uses and condition of the receiving water, including climate change considerations (e.g., potential increases in runoff and nutrient loading to receiving waters from increased storm events and watershed disturbances).
- Consideration of the best practical technology available to control phosphorus discharges.

10.2.1.2 Phosphorus Removal Criteria

A WWTP should be required to control the discharge of phosphorus if the following conditions exist:

- Eutrophication of the receiving water environment is either occurring or may occur (including climate change considerations), at a rate which may affect the existing and potential uses of the water environment.
- The wastewater effluent discharge is contributing or may contribute significantly to the rate of receiving water eutrophication.
- Required by the Regulatory Authority.

10.2.1.3 Method of Removal

Acceptable methods for phosphorus removal should include chemical precipitation, high-rate filtration, or biological processes.

10.2.1.4 Design Basis

10.2.1.4.1 Preliminary Testing

Laboratory, pilot, or full-scale studies of various chemical feed systems and treatment processes are recommended for existing plant facilities to determine the achievable performance level, cost-effective design criteria, and ranges of required chemical dosages.

The selection of a treatment process and chemical dosage for a new plant should be based on such factors as influent wastewater characteristics, effluent requirements, and anticipated treatment efficiency.

10.2.1.4.2 System Flexibility

Systems should be designed with sufficient flexibility to allow for several operational adjustments in chemical feed location, chemical feed rates, and for feeding alternate chemical compounds.

10.2.2 Process Requirements

10.2.2.1 Dosage

Typical chemical dosage requirements of various chemicals required for phosphorus removal are outlined in Table 10.1.

Table 10.1: Typical Chemical Dosage Requirements for Phosphorus Removal

Type of Treatment	Addition Point	Dosage Rate (mg/L)		
		Chemical	Range	Average
Mechanical				
Primary	Raw wastewater	Alum	100	100
		Ferric chloride	6 - 30	16
		Lime	167 - 200	185
Secondary	Raw wastewater	Alum		
		Ferric chloride	40 - 100	70
		Lime		
	Secondary section	Alum		
		Ferric chloride	30 - 150	65
		Lime	2 - 30	11
Waste Stabilization Ponds				
Seasonal retention ponds	Batch dosage to cells	Alum	100 - 210	163
		Ferric chloride	17 - 22	20
		Lime	250 - 350	300
Continuous discharge pond	Raw wastewater	Alum	225	225
		Ferric chloride	20	20
		Lime	400	400

Dosages will vary with the phosphorus concentration in the effluent. The required chemical dosage should include the amount needed to react with the phosphorus in the wastewater, the amount required to drive the chemical reaction to the desired state of completion, and the amount required due to inefficiencies in mixing or dispersion. Excessive chemical dosage should be avoided.

Alum generally is supplemented in a molar ratio in the range of a 1.4 to 2.5 mole Al/mole phosphorus. Ferric chloride is applied in a molar ratio of 2 to 4 mole ferric chloride/mole phosphorus for soluble phosphorus removal of greater than 90%. The precipitation of phosphorus can occur in a number of different locations during treatment processes:

- Before primary sedimentation (pre-precipitation).
- Before and following biological treatment (co-precipitation).
- Following secondary treatment (post-precipitation).
- At several locations in a process (split treatment).

Designers should be aware that dosage of some chemicals for phosphorous removal may consume available alkalinity. In some cases, alkalinity supplementation may be required to offset this consumption to maintain stable process operation. It is recommended that designers complete an alkalinity balance for the system considering available alkalinity in the influent, alkalinity consumed by biological nitrification processes (if any), alkalinity generated by biological denitrification processes (if any), and estimated alkalinity consumed by chemical phosphorous precipitation. It is desirable to maintain a net positive residual alkalinity to maintain stable process pH. If enough alkalinity is not available in the influent, chemical supplementation is required. Designers should be aware that the various chemicals used for phosphorus removal consume alkalinity to varying degrees in accordance with their respective chemical reaction equations. Choosing a different chemical for phosphorous precipitation may reduce or possibly eliminate the need for alkalinity supplementation.

10.2.2.2 Chemical Selection

The choice of lime or the salts of aluminum or iron should be based on the wastewater characteristics and the economics of the total system.

When lime is used it may be necessary to neutralize the high pH prior to subsequent treatment in secondary biological systems or prior to discharge in those flow schemes where lime treatment is the final step in the treatment process.

10.2.2.3 Chemical Feed System

In designing the chemical feed system for phosphorus removal, the following points should be considered:

- The need to select chemical feed pumps, storage tanks and piping suitable for use with the chosen chemical(s).
- Selection of chemical feed equipment with the required range in capacity.
- The need for a standby chemical feed pump.
- Provision of flow pacing for chemical pumps proportional to wastewater flowrates.
- Flexibility by providing a number of chemical application points.
- The need for protection of storage and piping from the effect of low temperatures.
- Selection of the proper chemical storage volume.
- The need for ventilation in chemical handling rooms.
- Provision for containment of any chemical spills.

10.2.2.4 Chemical Feed Points

Selection of chemical feed points should include consideration of the chemicals used in the process, necessary reaction times between chemical and polyelectrolyte additions, and the wastewater treatment processes and components utilized. Considerable flexibility in feed location should be provided, and multiple feed points are recommended.

10.2.2.5 Flash Mixing

Each chemical must be mixed rapidly and uniformly with the flow stream. Where separate mixing basins are provided, they should be equipped with mechanical mixing devices. The detention period should be at least 30 seconds.

10.2.2.6 Flocculation

The particle size of the precipitate formed by chemical treatment may be very small. Consideration should be given in the process design to the addition of synthetic polyelectrolytes to aid settling. The flocculation equipment should be adjustable to obtain optimum floc growth, control deposition of solids, and prevent floc destruction.

10.2.2.7 Liquid – Solids Separation

The velocity through pipes or conduits from flocculation basins to settling basins should not exceed 0.5 m/second in order to minimize floc destruction. Entrance works to settling basins should also be designed to minimize floc shear.

Settling basin design should be in accordance with criteria outlined in Chapter 6. For design of the sludge handling system, special consideration should be given to the type and volume of sludge generated in the phosphorus removal process.

10.2.2.8 Filtration
Effluent filtration should be considered where effluent phosphorus concentrations of less than 1 mg/L must be achieved.

10.2.3 Feed Systems

10.2.3.1 Location
All liquid chemical mixing and feed installations should be installed on corrosion resistant pedestals and elevated above the highest liquid level anticipated during emergency conditions.

Lime feed equipment should be located to minimize the length of slurry conduits. All slurry conduits should be accessible for cleaning.

10.2.3.2 Liquid Chemical Feed System
Liquid chemical feed pumps should be of the positive displacement type with variable feed rate. Pumps should be selected to feed the full range of chemical quantities required for the phosphorus mass loading conditions anticipated with the largest unit out of service. Screens and valves should be provided on the chemical feed pump suction lines.

An air break or anti-siphon device should be provided where the chemical solution stream discharges to the transport water stream to prevent an induction effect resulting in overfeed.

Consideration should be given to providing pacing equipment to optimize chemical feed rates.

10.2.3.3 Dry Chemical Feed System
Each dry chemical feeder should be equipped with a dissolver which can provide a minimum 5-minute retention at the maximum feed rate.

Polyelectrolyte feed installations should be equipped with two solution vessels and transfer piping for solution make-up and daily operation.

Make-up tanks should be provided with an eductor funnel or other appropriate arrangement for wetting the polymer during the preparation of the stock feed solution. Adequate mixing should be provided by a large-diameter low-speed mixer.

10.2.4 Storage Facilities

10.2.4.1 Size
Storage facilities should be sufficient to ensure that an adequate supply of the chemical is available at all times. The exact size required will depend on the size of the shipment, length of delivery time, and process requirements. Storage for a minimum of 10-day supply should be provided.

10.2.4.2 Location

The liquid chemical storage tanks and tank fill connections should be located within a containment structure having a capacity exceeding the total volume of all storage vessels. Valves on discharge lines should be located adjacent to the storage tank and within the containment structure.

Auxiliary facilities, including pumps and controls, within the containment area should be located above the highest anticipated liquid level. Containment areas should be sloped to a sump area and should not contain floor drains.

Bag storage should be located near the solution make-up point to avoid unnecessary transportation and housekeeping problems.

10.2.4.3 Accessories

Platforms, ladders, and railings should be provided as necessary to afford convenient and safe access to all filling connections, storage tank entries, and measuring devices.

Storage tanks should have reasonable access provided to facilitate cleaning.

10.2.5 Other Requirements

10.2.5.1 Materials

All chemical feed equipment and storage facilities should be constructed of materials resistant to chemical attack by all chemicals normally used for phosphorus treatment.

10.2.5.2 Temperature, Humidity, & Dust Control

Precautions should be taken to prevent chemical storage tanks and feed lines from reaching temperatures likely to result in freezing or chemical crystallization at the concentrations employed. A heated enclosure or insulation may be required. Consideration should be given to temperature, humidity, and dust control in all chemical feed room areas.

10.2.5.3 Cleaning

Consideration should be given to the accessibility of piping. Piping should be installed with plugged wyes, tees, or crosses at changes in direction to facilitate cleaning.

10.2.5.4 Drains & Draw-off

Above-bottom draw-off from chemical storage or feed tanks should be provided to avoid withdrawal of settled solids into the feed system. A bottom drain should also be installed for periodic removal of accumulated settled solids. Provisions should be made in the fill lines to prevent back siphonage of chemical tank contents.

10.2.6 Hazardous Chemical Handling

The requirements of Section 5.8.2 should be met.

10.2.7 Sludge Handling

10.2.7.1 General

Consideration should be given to the type and additional capacity of the sludge handling facilities needed when chemicals are added.

10.2.7.2 Dewatering

Design of dewatering systems should be based, where possible, on an analysis of the characteristics of the sludge to be handled. Consideration should be given to the ease of operation, effect of recycle streams generated, production rate, moisture content, de-water ability, final disposal, and operating cost.

10.3 Ammonia Removal

10.3.1 Breakpoint Chlorination

10.3.1.1 Applicability

The breakpoint chlorination process is best suited for removing relatively small quantities of ammonia, less than 5.0 mg/L NH_3-N, and in situations whose low residuals of ammonia or Total Nitrogen (TN) are required.

10.3.1.2 Design Considerations

10.3.1.2.1 Mixing

The reaction between ammonia and chlorine occurs instantaneously, and no special design features are necessary except to provide for complete uniform mixing of the chlorine with the wastewater. Good mixing can best be accomplished with in-line mixers or back mixed reactors. A minimum CT of 10 minutes is recommended.

10.3.1.2.2 Dosage

The sizing of the chlorine producing and/or feed device is dependent on the influent ammonia concentration to be treated as well as the degree of pre-treatment the wastewater has received. As the level of wastewater pre-treatment increases, the required amount of chlorine decreases and approaches the theoretical amount required to oxidize ammonia to nitrogen (7.6 mg Cl_2/L to 1 mg NH_3-N/L). A stoichiometric ratio of Cl_2 to NH_3-N = 7.6:1 will achieve a 95 to 99% conversion to N_2. Table 10.2 shows the quantities of chlorine required, based on operating experience as well as recommended design capabilities. These ratios are applied to the maximum anticipated influent ammonia concentration.

Table 10.2: Quantities of Chlorine Required for Three Wastewater Sources

Chlorine to NH_3 – N Ratio to Reach Breakpoint		
Wastewater Source	Experience	Recommended Design Capability
Raw	10 to 1	13 to 1
Secondary effluent	9 to 1	12 to 1
Lime settled and filtered secondary effluent	8 to 1	10 to 1

10.3.1.2.3 Monitoring

If insufficient chlorine is available to reach the breakpoint, no nitrogen will be formed, and the chloramines formed ultimately will revert back to ammonia. Provisions should be made to continuously monitor the waste, following chlorine addition, for free chlorine residual and to pace the chlorine feed device to maintain a set-point free chlorine residual.

10.3.1.2.4 Standby Equipment

The chemical feed assembly used for ammonia removal by breakpoint chlorination is considered in the preliminary design of the complete chlorination system, including those requirements for pre-chlorination, intermediate, and post-chlorination applications. Depending on the use of continuous chlorination at points within the system, some consideration is given to the use of standby chlorination equipment for the ammonia removal system. Reliability needs and maximum dosage requirements for the various application points should also be examined when sizing the equipment.

10.3.1.2.5 pH Adjustment

Except for wastewaters having a high alkalinity or treatment systems employing lime coagulation prior to chlorination, provisions should be made to feed an alkaline chemical to keep the pH of the wastewater in the proper range. A method for measuring and pacing the alkaline chemical feed pump to keep the pH in the desired range also should be provided.

10.3.2 Air Stripping

10.3.2.1 Applicability

The ammonia air stripping process is most economical if it is preceded by lime coagulation and settling. The ammonia stripping process can be used in a treatment system employing biological treatment or in a physical-chemical process. In most instances, more than 90% of the nitrogen in raw domestic wastewater is in the form of ammonia. The ammonia stripping process can be readily applied to most physical-chemical treatment systems, however, when the ammonia stripping process is to be preceded by a biological process, care must be exercised to ensure that nitrification does not occur in the secondary treatment process.

There is one serious limitation of the ammonia stripping process that should be recognized; namely, it is impossible to operate a stripping tower at air temperatures less than 0°C because of freezing within the tower. For treatment plants in cold weather locations, high pH stripping ponds may provide a simple solution to the problem of nitrogen removal.

10.3.2.2 Design Considerations

10.3.2.2.1 Tower Packing

Packings used in ammonia stripping towers may include 10 by 40 mm wood slats, plastic pipe, and a polypropylene grid. No specific packing spacing has been established. Generally, the individual splash should be spaced 40 to 100 mm horizontally and 50 to 100 mm vertically. A tighter spacing is used to achieve higher levels of ammonia removal and a more open spacing is used where lower levels of ammonia removal are acceptable. Considering the large volume of air required, towers should be designed for a total air headloss of less than 50 to 75 mm of water. Packing depths of 6.0 to 7.5 m should be used to minimize power costs.

10.3.2.2.2 Hydraulic Loadings

Allowable hydraulic loading is dependent on the type and spacing of the individual splash bars. Although HLRs used in ammonia stripping towers should range from 0.7 to 2.0 L/m^{2}s removal efficiency is significantly decreased at loadings exceeding 1.3 to 2.0 L/m^{2}s. The HLR should be such that a water droplet is formed at each individual splash bar as the liquid passes through the tower.

10.3.2.2.3 Air Requirements
Air requirements vary from 2,200 to 3,800 L/s for each L/s being treated in the tower. The 6 to 7.5 m of tower packing will normally produce a pressure drop of 15 to 40 mm of water.

10.3.2.2.4 Temperature
Air and liquid temperatures have a significant effect on the design of an ammonia stripping tower. Minimum operating air temperature and associated air density should be considered when sizing the fans to meet the desired air supply. Liquid temperature also affects the level of ammonia removal.

10.3.2.2.5 General Construction Features
The stripping tower may be either of the countercurrent (air inlet at base) or cross flow (air inlet along entire depth of fill) type. Generally, provisions should be made to have the capability to recycle tower effluent to increase the removal of ammonia nitrogen during cooler temperatures. Provisions should be made in the design of the tower structure and fill so that the tower packing is readily accessible or removable for removing possible deposits of calcium carbonate.

10.3.2.2.6 Process Control
During periods of tower operation when temperature, air, and wastewater flowrates, and scale formation are under control, the major process requirement necessary to ensure satisfactory ammonia removal is to control the influent pH. pH control should be practiced in the upstream lime-coagulation-settling process. This basin should be monitored closely to prevent excessive carryover of lime solids into the ammonia stripping process. Normal lime-addition required to raise the pH to 11.5 is 300 to 400 mg/L (as CaO).

10.4 Biological Nutrient Removal

10.4.1 Biological Phosphorus Removal

A number of biological phosphorus removal processes exist that have been developed as alternatives to chemical treatment. Phosphorus is removed in biological treatment by means of incorporating orthophosphate, polyphosphate, and organically bound phosphorus into cell tissue. The key to the biological phosphorus removal is the exposure of the microorganisms to alternating anaerobic and aerobic conditions. Exposure to alternating conditions stresses the microorganisms so that their uptake of phosphorus is above normal levels. Phosphorus is not only used for cell maintenance, synthesis, and energy transport but is also stored for subsequent use by the microorganisms. The sludge containing the excess phosphorus is either wasted or removed through a “sidestream” to release the excess. The alternating exposure to anaerobic and aerobic conditions can be accomplished in the main biological treatment process, or “mainstream” or in the return sludge stream, or “sidestream”.

10.4.1.1 Mainstream Phosphorus Removal (A/O Process)
The proprietary Anerobic and Oxic (A/O) process is a single sludge suspended-growth system that combines anaerobic stages and oxic stages (aerobic) in sequence. Settled sludge is returned to the influent end of the reactor and mixed with the incoming wastewater. Under anaerobic conditions, the phosphorus contained in the wastewater and the recycled cell mass is released as soluble phosphates. Some BOD reduction also occurs in this stage. The phosphorus is then taken up by the cell mass in the aerobic zone. Phosphorus is removed from the liquid stream in the WAS. The concentration of phosphorus in the effluent is dependent mainly on the ratio of BOD to phosphorus of the wastewater treated.

10.4.1.2 Sidestream Phosphorus Removal (PhoStrip Process)

In the proprietary PhoStrip process, a portion of the RAS from the biological treatment process is diverted to an anaerobic phosphorus stripping tank. The retention time in the stripping tank typically ranges from 8 to 12 hours. The phosphorus released in the stripping tank passes out of the tank in the supernatant, and the phosphorus-poor activated sludge is returned to the aeration tank. The phosphorus-rich supernatant is treated with lime or another coagulant in a separate tank and discharged to the primary sedimentation tanks or to a separate flocculation/clarification tank for solids separation. Phosphorus is removed from the system in the chemical precipitant. Conservatively designed PhoStrip and associated activated-sludge systems are capable of consistently producing an effluent with a total phosphorus content of less than 1.5 mg/L before filtration.

10.4.1.3 Design Criteria

Design criteria for biological Phosphorus removal is provided in Table 10.3.

Table 10.3: Design Criteria for Biological Phosphorus Removal

Design Parameter	Treatment Process		
	A/O	PhoStrip	SBR
F/M (kg BOD_5/kg MLVSS·d)	0.2 - 0.7	0.1 - 0.5	0.15 - 0.5
SRT (d)	2 - 25	10 - 30	
MLSS (mg/L)	2,000 - 4,000	600 - 5,000	2,000 - 3,000
HRT (hours)			
Anaerobic zone	0.5 - 1.5	8 - 12	1.8 - 3
Aerobic zone	1 - 3	4 - 10	1 - 4
RAS (% of influent flowrate)	25 - 40	20 - 50	N/A
Stripper underflow (% of influent flowrate)	N/A	10 - 20	N/A

10.4.2 Biological Nitrogen Removal

The principal nitrogen conversion and removal processes are conversion of ammonia nitrogen to nitrate by biological nitrification and removal of nitrogen by biological nitrification/denitrification.

10.4.2.1 Nitrification

Biological nitrification consists of the conversion of ammonia nitrogen to nitrite followed by the conversion of nitrite to nitrate. This process does not increase the removal of nitrogen from the waste stream over that achieved by conventional biological treatment. The principal effect is that nitrified effluent can be denitrified biologically. To achieve nitrification, all that is required is the maintenance of conditions suitable for the growth of nitrifying organisms.

Nitrification is also used when treatment requirements call for oxidation of ammonia-nitrogen. Nitrification may be carried out in conjunction with secondary treatment or in a tertiary stage. In each case, either suspended growth or attached growth reactors can be used.

10.4.2.1.1 Design Criteria

Design criteria for nitrification is provided in Table 10.4.

Table 10.4: Design Criteria for Nitrification

Design Parameter	Single Stage	Separate Stage
F/M (kg BOD_5/kg MLVSS·d)	0.12 - 0.25	0.05 - 0.2
SRT (d)	8 - 20	15 - 100
MLSS (mg/L)	1,500 - 3,500	1,500 - 3,500
HRT (hours)	6 - 15	3 - 6
RAS (% of influent flowrate)	50 - 150	50 - 200

10.4.2.2 Combined Nitrification/Denitrification

The removal of nitrogen by biological nitrification/denitrification is a 2-step process. In the first step, ammonia is converted aerobically to nitrate (NO_3^-) (nitrification). In the second step, nitrates are converted to nitrogen gas (denitrification).

The removal of nitrate by conversion to nitrogen gas can be accomplished biologically under anoxic conditions. The carbon requirements may be provided by internal sources, such as wastewater and cell material, or by an external source.

10.4.2.2.1 Bardenpho Process (Four-Stage)

The four-stage proprietary Bardenpho process uses both the carbon in the untreated wastewater and carbon from endogenous decay to achieve denitrification. Separate reaction zones are used for carbon oxidation and anoxic denitrification. The wastewater initially enters an anoxic denitrification zone to which nitrified mixed liquor is recycled from a subsequent combined carbon oxidation nitrification compartment. The carbon present in the wastewater is used to denitrify the recycled nitrate. Considering the organic loading is high, denitrification proceeds rapidly. The ammonia in the wastewater passes unchanged through the first anoxic basin to be nitrified in the first aeration basin. The nitrified mixed liquor from the first aeration basin passes into a second anoxic zone, where additional denitrification occurs using the endogenous carbon source. The second aerobic zone is relatively small and is used mainly to strip entrained nitrogen gas prior to clarification. Ammonia released from the sludge in the second anoxic zone is also nitrified in the last aerobic zone.

10.4.2.2.2 Oxidation Ditch

In an oxidation ditch, mixed liquor flows around a loop-type channel, driven and aerated by mechanical aeration devices. For nitrification/denitrification applications, an aerobic zone is established immediately downstream of the aerator, and an anoxic zone is created upstream of the aerator. By discharging the influent wastewater stream at the upstream end of the anoxic zone, some of the wastewater carbon source is used for denitrification. The effluent from the reactor is taken from the end of the aerobic zone for clarification. The system has only one anoxic zone, therefore, nitrogen removals are lower than those of the Bardenpho process.

10.4.3 Combined Biological Nitrogen & Phosphorus Removal

Several biological processes have been developed for the combined removal of nitrogen and phosphorus. Many of these are proprietary and use a form of the activated sludge process but employ combinations of anaerobic, anoxic, and aerobic zones or compartments to accomplish nitrogen and phosphorus removal.

10.4.3.1 A^2/O Process

The proprietary Anaerobic/Anoxic/Oxic (A^2/O) process provides an anoxic zone for denitrification with a detention period of approximately 1 hour. The anoxic zone is deficient in DO, but chemically bound oxygen in the form of nitrate or nitrite is introduced by recycling nitrified mixed liquor from the aerobic section. Effluent phosphorus concentrations of less than 2.0 mg/L can be expected without effluent filtration; with effluent filtration, effluent phosphorus concentrations may be less than 1.5 mg/L.

10.4.3.2 Phoredox Process (Five-Stage)

The proprietary Bardenpho process can be modified for combined nitrogen and phosphorus removal. The Phoredox modification of the Bardenpho process incorporates a fifth (anaerobic) stage for phosphorus removal. The 5-stage system provides anaerobic, anoxic, and aerobic stages for phosphorus, nitrogen, and carbon removal. A second anoxic stage is provided for additional denitrification using nitrate produced in the aerobic stage as the electron acceptor and the endogenous organic carbon as the electron donor. The final aerobic stage is used to strip residual nitrogen gas from solution and to minimize the release of phosphorus in the final clarifier. Mixed liquor from the first aerobic zone is recycled to the anoxic zone.

10.4.3.3 University of Cape Town Process

The University of Cape Town process eliminates RAS to the anoxic stage and the internal recycle is from the anoxic stage to the anaerobic stage. By returning the activated sludge to the anoxic stage, the introduction of nitrate to the anaerobic stage is eliminated, thereby improving the release of phosphorus in the anaerobic stage. The internal recycle feature provides for increased organic utilization in the anaerobic stage. The mixed liquor from the anoxic stage contains substantial soluble BOD but little nitrate. The recycle of the anoxic mixed liquor provides for optimal conditions for fermentation uptake in the anaerobic stage.

10.4.3.4 Design Criteria

Design criteria for combined biological nitrogen and phosphorus removal is provided in Table 10.5.

Table 10.5: Design Criteria for Combined Biological Nitrogen & Phosphorus Removal

Design Parameter	Treatment Process			
	A^2/O	Bardenpho (5-Stage)	University of Cape Town Process	SBR
F/M (kg BOD_5/kg MLVSS·d)	0.15 - 0.25	0.1 - 0.2	0.1 - 0.2	0.1
SRT (d)	4 - 27	10 - 40	10 - 30	20 - 40
MLSS (mg/L)	2,000 - 4,000	2,000 - 5,000	2,000 - 5,000	600 - 5,000
HRT (hours) Anaerobic zone Anoxic zone - 1 Aerobic zone - 1 Anoxic zone - 2 Aerobic zone - 2 Settle/decant **Total**	0.5 - 1.5 0.5 - 1 3.5 - 6 **4.8 - 8.5**	1 - 2 2 - 4 4 - 12 2 - 4 0.5 - 1 **9.5 - 23**	1 - 2 2 - 4 4 - 12 **7.0 - 18.0**	Batch times 1.5 - 3 1 - 3 2 - 4 3.5 - 10
RAS (% of influent flowrate)	25 - 50	80 - 100	80 - 100	-
Internal recycle (% of influent flowrate)	100 - 300	400 - 600	100 - 600 (anoxic) & 50 - 100 (aerobic)	-

10.4.4 Sequencing Batch Reactor

The SBR can be operated to achieve any combination of carbon oxidation, nitrogen reduction, and phosphorus removal. Reduction of these constituents can be accomplished with or without chemical addition by changing the operation of the reactor. Phosphorus can be removed by coagulant addition or biologically without coagulant addition. By modifying the reaction times, nitrification or nitrogen removal can also be accomplished. Overall cycle time may vary from 3 to 24 hours. A carbon source in the anoxic phase is required to support denitrification-either an external source or endogenous respiration of the existing biomass.

10.4.5 Detailed Design Manuals

The following sources contain detailed design information for biological nutrient removal:

- *Nutrient Control Manual of Practice FD-7* from the Water Environment Federation.
- *Design Manual: Phosphorus Removal* from the United States Environment Protection Agency.
- *Process Design Manual for Nitrogen Control* from the United States Environment Protection Agency.
- *Process Design Manual for Phosphorus Removal* from the United States Environment Protection Agency.
- *Treatment Processes for the Removal of Ammonia from Municipal Wastewater* from Environment and Climate Change Canada.

10.5 Effluent Filtration

10.5.1 General

10.5.1.1 Applicability

Effluent filtration is generally necessary when effluent quality better than 15 mg/L BOD_5, 15 mg/L SS, and 1.0 mg/L phosphorus is required.

Where effluent SS requirements are less than 10 mg/L, where secondary effluent quality can be expected to fluctuate significantly, or where filters follow a treatment process where significant amounts of algae will be present, a pre-treatment process such as chemical coagulation and sedimentation or other acceptable process should precede the filter units.

10.5.1.2 Design Considerations

Factors to consider when choosing between the different filtration systems which are available, include the following:

- The installed capital and expected operating and maintenance costs.
- The energy requirements of the systems (head requirements).
- The media types and sizes and expected solids capacities and treatment efficiencies of the system.
- The backwashing systems, including type, backwash rate, backwash volume, effect on wastewater works, etc.

Care should be given in the selection of pumping equipment ahead of filter units to minimize shearing of floc particles. Consideration should be given in the plant design to providing flow-equalization facilities to moderate filter influent quality and quantity.

10.5.2 Location of Filter Systems

Effluent filtration should precede the chlorine contact chamber to minimize chlorine usage, to allow more effective disinfection and to minimize the production of chloro-organic compounds.

To allow excessive biological growths and grease accumulations to be periodically removed from the filter media, a chlorine application point should be provided upstream of the filtration system (chlorine would only be dosed as necessary at this location with dechlorination used as required to ensure protection of aquatic life).

10.5.3 Number of Units

Total filter area should be provided in two or more units, and the filtration rate should be calculated on the total available filter area with one unit out of service.

10.5.4 Filter Types

Filters may be of the gravity type or pressure type. Pressure filters should be provided with ready and convenient access to the media for treatment or cleaning. Where greases or similar solids, which result in filter plugging are expected, filters should be of the gravity type.

10.5.5 Filtration Rates

10.5.5.1 Hydraulic Loading Rate

Filtration rates at peak hourly wastewater flowrates, including backwash flows, should not exceed 2.1 $L/m^2 \cdot s$ for shallow bed single media systems (if raw wastewater flow equalization is provided, lower peak filtration rates should be used in order to avoid under-sizing of the filter).

Filtration rates at peak hourly wastewater flowrates, including backwash flows, should not exceed 3.3 $L/m^2 \cdot s$ for deep bed filters (if raw wastewater flow equalization is provided, lower peak filtration rates should be used to avoid undersizing of the filter). The manufacturer's recommended maximum filtration rate should, however, not be exceeded.

10.5.5.2 Organic Loading Rate

Peak solids loading rate should not exceed 50 $mg/m^2 \cdot s$ for shallow bed filters and 80 $mg/m^2 \cdot s$ for deep bed filters (if raw wastewater flow equalization is provided, lower peak solids loading rates should be used to avoid undersizing of the filter).

10.5.6 Backwash

10.5.6.1 Backwash Rate

The backwash rate should be adequate to fluidize and expand each media layer a minimum of 20% based on the media selected. The backwash system should be capable of providing a variable backwash rate so that the maximum rate is at least 14 $L/m^2 \cdot s$ and a minimum backwash period of 10 minutes.

10.5.6.2 Backwash

Pumps for backwashing filter units should be sized and interconnected to provide the required rate to any filter with the largest pump out of service. Filtered water should be used as the source of backwash water. Waste filter backwash should be adequately treated.

Air scour or mechanical agitation systems to improve backwash effectiveness are recommended.

If instantaneous backwash rates represent more than 15% of the average daily design flowrate of the plant, a backwash holding tank should be provided to equalize the flow of backwash water to the plant.

10.5.7 Filter Media

10.5.7.1 Selection

Selection of proper media size will depend on the filtration rate selected, the type of treatment provided prior to filtration, filter configuration, and effluent quality objectives. In dual or multi-media filters, media size selection must consider compatibility among media.

10.5.7.2 Media Specifications

Table 10.6 provides minimum media depths and the normally acceptable range of media sizes. The designer has the responsibility for selection of media to meet specific conditions and treatment requirements relative to the project under consideration.

Table 10.6: Media Depths & Sizes

Media Type	Single Media	Multi-Media	
		2 Media	3 Media
Anthracite	-	50 cm 1.0 - 2.0 mm	50 cm 1.0 - 2.0 mm
Sand	120 cm 1.0 - 4.0 mm	30 cm 0.5 - 1.0 mm	25 cm 0.6 - 0.8 mm
Garnet or similar material	-	-	5 cm 0.3 - 0.6 mm
Minimum Depth Effective Size			

The uniformity coefficient should be 1.7 or less.

10.5.8 Filter Appurtenances

The filters should be equipped with:

- Washwater troughs.
- Surface wash or air scouring equipment.
- A means of measurement and positive control of the backwash rate.
- Equipment for measuring filter head loss.
- Positive means of shutting off flow to a filter being backwashed.
- Filter influent and effluent sampling points.

If automatic controls are provided, there should be a manual override for operating equipment, including each individual valve essential to the filter operation. The underdrain system should be designed for uniform distribution of backwash water (and air, if provided) without danger of clogging from solids in the backwash water. Provisions should be made to allow periodic chlorination of the filter influent or backwash water to control slime growths. If air is to be used for filter backwash, separate backwash blowers should be provided.

10.5.9 Reliability

Each filter unit should be designed and installed so that there is ready and convenient access to all components and the media surface for inspection and maintenance without taking other units out of service. The need for housing of filter units should depend on expected extreme climatic conditions at the treatment plant site. As a minimum, all controls should be enclosed. The structure housing filter controls and equipment should be provided with adequate heating and ventilation equipment to minimize problems with excess humidity.

10.5.10 Backwash Surge Control

The rate of return of waste filter backwash water to treatment units should be controlled such that the rate does not exceed 15% of the design average daily flowrate to the treatment units. The hydraulic and organic load from waste backwash water should be considered in the overall design of the treatment plant. Surge tanks should have a minimum capacity of two backwash volumes, although additional capacity should be considered to allow for operational flexibility. Where waste backwash water is returned for treatment by pumping, adequate pumping capacity should be provided with the largest unit out of service.

10.5.11 Backwash Water Storage

Total backwash water storage capacity provided in an effluent clearwell or other unit should equal or exceed the volume required for two complete backwash cycles.

10.5.12 Proprietary Equipment

Where proprietary filtration equipment not conforming to the preceding requirements is proposed, data which supports the capability of the equipment to meet effluent requirements under design conditions should be provided. Such equipment will be reviewed on a case-by case basis at the discretion of the Regulatory Authorities.

10.6 Microscreening

10.6.1 General

10.6.1.1 Applicability

Microscreening units may be used following a biological treatment process for the removal of residual SS. Selection of this unit process should consider final effluent requirements, the preceding biological treatment process, and anticipated consistency of the biological process to provide a high-quality effluent.

10.6.1.2 Design Considerations

Pilot plant testing on existing secondary effluent is encouraged. Where pilot studies so indicate, where microscreens follow trickling filters or ponds, or where effluent SS requirements are less than 10 mg/L, a pre-treatment process such as chemical coagulation and sedimentation should be provided. Care should be taken in the selection of pumping equipment ahead of microscreens to minimize shearing of floc particles. The process design should include flow equalization facilities to moderate microscreen influent quality and quantity.

10.6.2 Screen Material

The microfabric should be a material demonstrated to be durable through long-term performance data. The aperture size must be selected considering required removal efficiencies, normally ranging from 20 to 35 µm. The use of pilot plant testing for aperture size selection is recommended.

10.6.3 Screening Rate

The screening rate should be selected to be compatible with available pilot plant test results and selected screen aperture size but should not exceed 3.4 $L/m^2 \cdot s$ of effective screen area based on the maximum hydraulic flowrate applied to the units. The effective screen area should be considered as the submerged screen surface area less the area of screen blocked by structural supports and fasteners. The screening rate should be that applied to the units with one unit out of service.

10.6.4 Backwash

All waste backwash water generated by the microscreening operation should be recycled for treatment. The backwash volume and pressure should be adequate to assure maintenance of fabric cleanliness and flow capacity. Equipment for backwash of at least 1.65 L/m·s of screen length and 4.22 kgf/cm^2, respectively, should be provided. Backwash water should be supplied continuously by multiple pumps, including one standby, and should be obtained from microscreened effluent. The rate of return of waste backwash water to treatment units should be controlled such that the rate does not exceed 15% of the design average daily flowrate to the treatment plant. The hydraulic and organic load from waste backwash water should be considered in the overall design of the treatment plant. Where waste backwash is returned for treatment by pumping, adequate pumping capacity should be provided with the largest unit out of service. Provisions should be made for measuring backwash flow.

10.6.5 Appurtenances

Each microscreen unit should be provided with automatic speed controls with provisions for manual override, a bypass weir with an alarm for use when the screen becomes blinded to prevent excessive head development and means for dewatering the unit for inspection and maintenance. Bypassed flows must be segregated from water used for backwashing. Equipment for control of biological slime growths should be provided. The use of chlorine should be restricted to those installations where the screen material is not subject to damage by the chlorine.

10.6.6 Reliability

A minimum of two microscreen units should be provided, each unit being capable of independent operation. A supply of critical spare parts should be provided and maintained. All units and controls should be enclosed in a heated and ventilated structure with adequate working space to provide for ease of maintenance.

10.7 Activated Carbon Adsorption

10.7.1 Applicability

In tertiary treatment, the role of activated carbon is to remove the relatively small quantities of refractory organics, as well as inorganic compounds such as nitrogen, sulphides, and heavy metals, remaining in an otherwise well-treated wastewater.

Activated carbon may also be used to remove soluble organics following chemical-physical treatment.

10.7.2 Design Considerations

The usefulness and efficiency of carbon adsorption for wastewater treatment depends on the quality and quantity of the delivered wastewater. To be fully effective, the carbon unit should receive an effluent of uniform quality, without surges in the flow. Other wastewater qualities of concern include SS, oxygen demand, other organics such as Methylene Blue Active Substance (MBAS) or phenol and DO. Environmental parameters of importance include pH and temperature. Consideration also should be given to the type of activated carbon available. Activated carbons produced from different base materials and by different activation processes will have varying adsorptive capacities. Some factors influencing adsorption at the carbon/liquid interface are:

- Attraction of carbon for solute.
- Attraction of carbon for solvent.
- Solubilizing power of solvent or solute.
- Association.
- Ionization.
- Effect of solvent on orientation at interface.
- Competition for interface in presence of multiple solutes.
- Adsorption.

- Molecular size of molecules in the system.
- Pore size distribution in carbon.
- Surface area of carbon.
- Concentration of constituents.

There are several different activated carbon contactor systems that can be selected. The carbon columns can be either of the pressure or gravity type.

10.7.3 Unit Sizing

10.7.3.1 Contact Time

The CT should be calculated based on the volume of the column occupied by the activated carbon. Generally, carbon CTs of 15 to 35 minutes are used depending on the application, the wastewater characteristics, and the desired effluent quality. For tertiary treatment applications, carbon CTs of 15 to 20 minutes should be used where the desired effluent quality is a COD of 10 to 20 mg/L, and 30 to 35 minutes when the desired effluent COD is 5 to 15 mg/L. For chemical-physical treatment plants, carbon CTs of 20 to 35 minutes should be used, with a CT of 30 minutes being typical.

10.7.3.2 Hydraulic Loading Rate

Hydraulic loading rates of 2.5 to 7.0 L/m^2·s of cross section of the bed should be used for upflow carbon columns. For downflow carbon columns, HLRs of 2.0 to 3.3 L/m^2·s are used. Actual operating pressure seldom rises above 7.0 kN/m^2 for each 0.3 m of bed depth.

10.7.3.3 Depth of Bed

The depth of bed will vary considerably, depending primarily on carbon CT, and may be from 3 to 12 m. A minimum carbon depth of 3.0 m is recommended. Typical total carbon depths range from 4.5 to 6 m. Freeboard has to be added to the carbon depth to allow an expansion of 10 to 50% for the carbon bed during backwash or for expanded bed operation. Carbon particle size and water temperature will determine the required quantity of backwash water to attain the desired level of bed expansion.

10.7.3.4 Number of Units

A minimum of two parallel carbon contactor units are recommended for any size plant. A sufficient number of contactors should be provided to ensure an adequate carbon CT to maintain effluent quality while one column is offline during removal of spent carbon for regeneration or for maintenance.

10.7.4 Backwashing

The rate and frequency of backwash is dependent on hydraulic loading, the nature and concentration of SS in the wastewater, the carbon particle size, and the method of contacting. Backwash frequency can be prescribed arbitrarily (each day at a specified time), or by operating criteria (headloss or turbidity). Duration of backwash may be 10 to 15 minutes.

The normal quantity of backwash water employed is less than 5% of the product water for a 0.8 m deep filter and 10 to 20% for a 4.5 m filter.

Recommended backwash flowrates for granular carbons of 8 x 12 or 12 x 30-mesh are 8 to 14 L/m^2·s.

10.7.5 Valve & Pipe Requirements

Upflow units should be piped to operate either as upflow or downflow units as well as being capable of being backwashed. Downflow units should be piped to operate as downflow and in series. Each column must be valved to be backwashed individually. Furthermore, downflow series contactors should be valved and piped so that the respective position(s) of the individual contactors can be interchanged.

10.7.6 Instrumentation

The individual carbon columns should be equipped with flow and headloss measuring devices.

10.7.7 Hydrogen Sulphide Control

Methods that can be incorporated into the plant design to cope with hydrogen sulphide production include:

- Providing upstream biological treatment to satisfy as much of the biological oxygen demand as possible prior to carbon treatment.
- Reducing detention time in the carbon columns based on DO concentrations of the effluent.
- Backwashing the columns at more frequent intervals.
- Chlorinating carbon column influent.
- In upflow expanded beds the introducing of an oxygen source, such as air or hydrogen peroxide, to keep the columns aerobic.

10.7.8 Carbon Transport

Provisions must be made to remove spent carbon from the carbon contactors. It is important to obtain a uniform withdrawal of carbon over the entire horizontal surface area of the carbon bed. Care must be taken to ensure that gravel or stone supporting media used in downflow contactors does not enter the carbon transport system.

Activated carbon should be transported hydraulically. Carbon slurries can be transported using water or air pressure, centrifugal or diaphragm pumps, or eductors. The type of motive equipment selected requires a balance of System Owner preference, column control capabilities, capital, and maintenance costs, and pumping head requirements.

Carbon slurry piping systems should be designed to provide approximately 8.0 L of transport water for each kg of carbon removed. Pipeline velocities of 0.9 to 1.5 m/s are recommended.

Long-radius elbows or tees and crosses with cleanouts should be used at points of pipe direction change. Valves should be of the ball or plug type. No valves should be installed in the slurry piping system for the purpose of throttling flows.

10.7.9 Carbon Regeneration

10.7.9.1 Quantities of Spent Carbon

The carbon dose used to size the regeneration facilities depends on the strength of the wastewater applied to the carbon and the required effluent quality. Typical carbon dosages that might be anticipated for municipal wastewaters are shown in Table 10.7.

Table 10.7: Typical Carbon Dosages for Different Column Wastewater Influents

Pre-treatment	Typical Carbon Dosage Required Per m^3 of Column Throughput (g/m^3)*
Coagulated, settled, and filtered activated sludge effluent.	35 - 70
Filtered secondary effluent.	70 - 100
Coagulated, settled, and filtered raw wastewater (physical-chemical).	100 - 300

*Loss of carbon during each regeneration cycle typically will be 5 to 10%. Make-up carbon is based on carbon dosage and the quality of the regenerated carbon.

10.7.9.2 Carbon Dewatering

Dewatering of the spent carbon slurry prior to thermal regeneration may be accomplished in spent carbon drain bins. The drainage bins should be equipped with screens to allow the transport of water to flow from the carbon. Two drain bins should be provided.

Dewatering screws may also be used to dewater the activated carbon. A bin must be included in the system to provide a continuous supply of carbon to the screw, as well as maintain a positive seal on the furnace.

10.7.9.3 Regeneration Furnace

Partially dewatered carbon may be fed to the regeneration furnace with a screw conveyor equipped with a variable speed drive to control the rate of carbon feed precisely.

The theoretical furnace capacity is determined by the anticipated carbon dosage. An allowance for furnace downtime on the order of 40% should be added to the theoretical capacity.

Based on the experience gained from two full-scale facilities, provisions should be made to add approximately 1.0 kg of steam per kg of carbon regenerated. Fuel requirements for the carbon regeneration furnace are 7,000 kJ/kg of carbon when regenerating spent carbon on tertiary and secondary effluent applications. To this value, the energy requirements for steam and an afterburner, if required, must be added.

The furnace should be designed to control the carbon feed rate, rabble arm speed, and hearth temperatures. The off-gases from the furnace must be within acceptable air pollution standards. Air pollution control equipment should be designed as an integral part of the furnace and include a scrubber for removing carbon fines and an afterburner for controlling odours.

10.8 Constructed Wetlands

10.8.1 General

Constructed wetlands are inundated land areas with water depths typically less than 0.6 m that support the growth of emergent plants such as cattail, bulrush, reeds, and sedges. The vegetation provides surface for the attachment of bacterial films, aids in the filtration and adsorption of wastewater constituents, transfers oxygen into the water column, and controls the growth of algae by restricting the penetration of sunlight.

Although plant uptake is an important consideration in contaminant removal, particularly nutrient removal, it is only one of many active removal mechanisms in the wetland environment. Removal mechanisms have been classified as physical, chemical, and biological and are operative in the water column, the humus and soil column beneath the growing plants, and at the interface between the water and soil columns. Considering most of the biological transformations take place on or near a surface to which bacteria are attached, the presence of vegetation and humus is very important. Wetland systems are designed to provide maximum production of humus material through profuse plant growth and organic matter decomposition.

For any work being done in or near a wetland, System Owner should refer to Regulatory Authority on potential permitting/approval requirements. This also applies when creating a new wetland.

10.8.2 Types

Wastewater treatment systems using constructed wetlands have been categorized as either Free Water Surface (FWS) or Subsurface Flow (SF) types:

- Free water surface wetlands.
- Subsurface flow.

A FWS system consists of basins or channels with a natural or constructed subsurface barrier to minimize seepage. Emergent vegetation is grown and wastewater is treated as it flows through the vegetation and plant litter. Free water surface wetlands are typically long and narrow to minimize short-circuiting.

A SF wetland system consists of channels or basins that contain gravel or sand media which will support the growth of emergent vegetation. The bed of impermeable material is sloped typically between 0 to 2%. Wastewater flows horizontally through the root zone of the wetland plants about 100 to 150 mm below the gravel surface. Treated effluent is collected in an outlet channel or pipe.

10.8.3 Site Evaluation

Site characteristics that must be considered in wetland system design include topography, soil characteristics, existing land use, flood hazard, and climate (including climate change considerations).

10.8.3.1 Topography

Level to slightly sloping, uniform topography is preferred for wetland sites because FWS are generally designed with level basins or channels, and SF systems are normally designed and constructed with slopes of 1% or slightly more. Although basins may be constructed on steeper sloping or uneven sites, the amount of earthwork required will affect the cost of the system. Thus, slope gradients should be less than 5%.

10.8.3.2 Soil Characteristics

Sites with slowly permeable (< 1.4×10^{-4} cm/second) surface soils or subsurface layers are most desirable for wetland systems because the objective is to treat the wastewater in the water layer above the soil profile, therefore, percolation losses through the soil profile should be minimized. As with overland-flow systems, the surface soil will tend to seal with time due to deposition of solids and growth of bacterial slimes. Permeabilities of native soils may be purposely reduced by compacting during construction. Sites with high permeability soils may be used for small systems by constructing basins with clay or artificial liners. The depth of soil to groundwater should be a minimum of 0.3 to 0.6 m to allow sufficient distance for treatment of any percolate entering the groundwater.

10.8.3.3 Flood Hazard

Wetland sites should be located outside of flood plains, or protection from flooding should be provided. Considerations for the impacts of climate change, such as potential changes to flood extents, should be accounted for (refer to Chapter 2 for further guidance).

10.8.3.4 Existing Land Use

Open space or agricultural lands, particularly those near existing natural wetlands, are preferred for wetland sites. Constructed wetlands can enhance existing natural wetlands by providing additional wildlife habitat and, in some cases, by providing a more consistent water supply.

10.8.3.5 Climate

The use of wetland systems in cold climates is possible. Considering the principal treatment systems are biological, treatment performance is strongly temperature sensitive. Storage will be required where treatment objectives cannot be met due to low temperatures. Climate change may be considered in design where it may worsen projected low temperatures.

10.8.4 Pre-Application Treatment

Artificial wetlands may be designed to accept wastewater with minimal (coarse screening and comminution) pre-treatment, however, the level of pre-treatment will influence the quality of the final effluent and, therefore, overall treatment objectives must be considered. Since there is no permanent escape mechanism for phosphorus within the wetland, phosphorus reduction by chemical addition is also recommended as a pre-treatment step to ensure continued satisfactory phosphorus removal within the marsh.

10.8.5 Vegetation Selection & Management

The plants most frequently used in constructed wetlands include cattails, reeds, rushes, bulrushes, and sedges. All of these plants are ubiquitous and tolerate freezing conditions. The important characteristics of the plants related to design are the optimum depth of water for FWS systems and the depth of rhizome and root systems for SF systems. Cattails tend to dominate in water depths over 0.15 m. Bulrushes grow well at depths of 0.05 to 0.25 m. Reeds grow along the shoreline and in water up to 1.5 m deep but are poor competitors in shallow waters. Sedges normally occur along the shoreline and in shallow water than bulrushes. Cattail rhizomes and roots extend to a depth of approximately 0.3 m, whereas reeds extend to more than 0.6 m and bulrushes to more than 0.75 m. Reeds and bulrushes are normally selected for SF systems because the depth of rhizome penetration allows for the use of deeper basins.

Harvesting of wetland vegetation is generally not required, especially for SF systems, however, dry grasses in FWS systems are burned off periodically to maintain free-flow conditions and to prevent channeling of the flow. Removal of the plant biomass for the purpose of nutrient removal is normally not practical.

10.8.6 Design Parameters

10.8.6.1 Detention Time

10.8.6.1.1 Free Water Surface Wetlands

The relationship between BOD removal and detention times for FWS is represented by the equation:

$$C_e = Co\exp(-K_T t)$$

Where:

C_e = Effluent BOD (mg/L).
C_o = Influent BOD (mg/L).
k_T = Temperature dependent rate constant (d^{-1}).
k_T = k_{20} x $1.06^{(T-20)}$.
k_{20} = d^{-1}.
T = Average monthly water temperature (°C).
t = Average detention time (d).

$$A_s cy/Q_a$$

Where:

A_s = Design surface area of wetland (m^2).
c = Fraction of cross-sectional area not used by plants.
y = Depth of water in the wetland (m).
Q_A = Average flow through the wetland ([(Q_{in} + Q_{out})/2], m^3/d).

Note: See Table 10.9 for typical parameters for FWS and SF wetlands.

10.8.6.1.2 Subsurface Flow Wetlands

The relationship between BOD removal and detention times for SF is represented by the equation:

$$C_e = Co\exp(-K_T t')$$

Where:

C_e = Effluent BOD (mg/L).
C_o = Influent BOD (mg/L).
k_T = Temperature dependent rate constant (d^{-1}).
K_T = k_{20} x $1.06^{(T-20)}$.
k_{20} = d^{-1}.

$$t' = A_s \alpha y / Q_A$$

Where:

A_s	=	Design surface area of wetland (m^2).
α	=	Porosity of basin medium (see Table 10.8 for media characteristics).
y	=	Depth of water in the wetland (m).
Q_A	=	Average flow through the wetland ([(Q_{in} + Q_{out})/2], m^3/d).

Note: See Table 10.9 for typical parameters for FWS and SF wetlands.

10.8.6.2 Water Depth

For FWS, the design water depth depends on the optimum depth for the selected vegetation. In cold climates, the operating depth is normally increased in the winter to allow for ice formation on the surface and to provide the increased detention time required at colder temperatures. Systems should be designed with an outlet structure that allows for varied operating depths.

The design depth of SF systems is controlled by the depth of penetration of the plant rhizomes and roots because the plants supply oxygen to the water through the root/rhizome system.

See Table 10.9 for typical FWS and SF water depths.

10.8.6.3 Hydraulics & Hydrological Considerations

Manning's equation is generally accepted as a model for the flow of water through FWS wetland systems. Flow velocity depends on depth of the water, hydraulic gradient (i.e., slope of the water surface), and the resistance to flow.

$$v = (1/n)\left(y^{2/3}\right)\left(s^{1/2}\right)$$

Where:

v	=	Flow velocity (m/s).
n	=	Manning's coefficient (s/$m^{1/3}$).
s	=	Hydraulic gradient (m/m).
y	=	Water depth (m).

The relationship between Manning's *n* coefficient and the resistance factor is defined as:

$$n = a/y^{1/2}$$

Where:

a = the resistance factor ($s \cdot m^{1/6}$).

Reed et al. (1995) presented the following values for a in FWS wetlands.

Sparse, low standing vegetation (y > 0.4 m):
a = 0.4 $s \cdot m^{1/6}$

Moderately dense vegetation (y ≥ 0.3 m):
a = 1.6 $s \cdot m^{1/6}$

Very dense and litter (y < 0.3 m)
a = 6.4 $s \cdot m^{1/6}$

The aspect ratio (i.e., length to width ratio) selected for a FWS wetland can influence the hydraulic regime because resistance to flow increases as length increases. Reed et al. (1995) developed a model that can estimate the maximum desirable length of an FWS wetland channel.

$$L = [(A_s)(y^{2.667})(m^{0.5})86400/(a)(Q_A)]^{0.667}$$

Where:

L = Maximum length of wetland cell (m).
A_s = Design surface area of wetland (m^2).
y = Depth of water in the wetland (m).
m = Portion of available hydraulic gradient used to provide the necessary head (percent as a decimal).
A = Resistance factor ($s \cdot m^{1/6}$).
Q_A = Average flow through the wetland ($[(Q_{in} + Q_{out})/2]$, m^3/d).

An initial *m* value between 10 and 20% is suggested for design to ensure a future reserve as a safety factor. In the general case this model produces an aspect ratio of 3-to-1 or less. Using the average flow $_{QA}$ in the model compensates for the influence of precipitation, evapotranspiration, and seepage on the flow through the wetland. Climate change should be considered when determining climatic conditions and processes (e.g., precipitation events and evapotranspiration rates). The design surface area is the bottom area of the wetland.

Darcy's Law describes the flow regime in a porous media and is generally accepted for the hydraulic design of SF wetlands.

Because:

$$v = k_s s = Q_A / A_c Y$$

Therefore:

$$Q_A = K_s A_c s$$

Where:

Q_A = Average flow through the SF wetland (m^3/d).
K_s = Hydraulic conductivity of a unit area of the wetland perpendicular to the flow direction ($m^3/m^2/d$).
A_c = Total cross-sectional area perpendicular to flow (m^2).
s = Hydraulic gradient or slope of the water surface in the wetland (m/m).
v = Darcy's velocity, the apparent flow velocity through the-cross sectional area.

The aspect ratio (i.e., length to width ratio) selected for a SF wetland can influence the hydraulic regime because resistance to flow increases as length increases. Reed et al. (1995) developed a model that can estimate the maximum desirable length of an SF wetland channel.

$$W = (1/y)[(Q_A)(A_s)/(m)(k_s)]^{0.5}$$

Where:

W = Maximum width of the SF wetland cell (m).
A_s = Design surface area of wetland (m^2).
y = Depth of water in the wetland (m).
m = Portion of available hydraulic gradient used to provide the necessary head (percent as a decimal).
k_s = Hydraulic conductivity of the media used ($m^3/m^2/d$).
Q_A = Average flow through the wetland (m^3/d).

$$\frac{(Q_{IN} + Q_{OUT})}{2}$$

The *m* value in the equation above ranges from 5 to 20% of the potential head available. For large projects, the hydraulic conductivity (k_s) should be directly measured with a sample of the media to be used. When using the maximum width equation, not more than 1/3 of the effective hydraulic conductivity (k_s) should be used in the calculation, and *m* value should not exceed 20% to ensure a large safety factor against potential clogging and other contingencies not defined at the time of design. Table 10.8 gives the typical characteristics for media in SF wetlands.

Table 10.8: Typical Media Characteristics for Subsurface Flow Wetlands

Media type	D_{10} Effective size (mm)	Porosity (α)	K_s ($m^3/m^2 \cdot d$)
Course sand	2	0.28 - 0.32	100 - 1,000
Gravelly sand	8	0.30 - 0.35	500 - 5,000
Fine gravel	16	0.35 - 0.38	1,000 - 10,000
Medium gravel	32	0.36 - 0.40	10,000 - 50,000
Coarse rock	128	0.38 - 0.45	50,000 - 250,000

Table 10.9 provides typical parameters for FWS and SF wetlands.

Table 10.9: Typical Parameters for Free Water Surface & Subsurface Flow Wetlands

Parameter	FWS Wetland	SF Wetlands
Porosity (α)	0.65 - 0.75	0.35 - 0.45
Depth (y) (m)	0.15 - 0.60	0.30 - 0.60
Fraction of cross-sectional area not used by plants (c)	0.65 - 0.75	0.65 - 0.75
BOD_5 Removal		
K_{20}, d^{-1}	0.678	1.104
θ	1.06	1.06
Background concentration (mg/L)	6	6
TSS Removal		
C_e/C_o	[0.1139 + 0.00213(HLR)]	[0.1058 + 0.0011(HLR)]
HLR (mm/d x 0.1)	-	-
TSS removal does not depend on temperature		
Background concentration (mg/L)	6	6
Ammonia Removal		
At 0°C ($K_T(d^{-1})$)	0	0
At 1°C (K_{20})	0.2187	$(K_{NH})(\theta)^{(T-20)}$
θ	1.048	1.048
K_{NH} = rate constant 20°C for SF wetlands, d^{-1} (*rz* = portion of SF bed occupied by plant roots, % as a decimal can equal 0 to 0.1, depending on root depth (0.5 is typical)	-	$K_{NH} = 0.1854 + 0.3922(rz)^{(2.6077)}$
Background concentration	0.2	0.2
Note: it is prudent to assume that all TKN entering the wetland can appear as ammonia; so, assume C_o for ammonia is equal to influent TKN.		
Nitrate Removal		
At 0°C ($K_T(d^{-1})$)	0	0
At 1°C + (K_{20})	1.0	1.0
θ	1.15	1.15
Background concentration (mg/L)	0.20	0.20
Note: it is conservative to assume that all ammonia removed in the previous step can appear as nitrate; so, C_o for nitrate removal design equals C_e from ammonia removal plus nitrate present in the influent.		

Parameter	FWS Wetland	SF Wetlands
TN Removal		
Effluent $T_N = C_{e(NO3)} + (C_{e(NH4)} – C_{e(NO3)})$		
Background concentration (mg/L)	0.4	0.4
Note: a specific model for TN removal is not available in this set. The effluent TN can be estimated as the sum of residual ammonia and remaining nitrate ($C_o – C_e$).		
Total Phosphorus Removal		
	C_e/C_o = exp($-K_p$/HLR)	C_e/C_o = exp($-K_p$/HLR)
K_p, mm/d x 0.1	2.73	2.73
TP removal does not depend on temperature		
HLR = average HLR (cm/d)		
Background concentration (mg/L)	0.5	0.5
Fecal Coliform Removal		
C_e/C_o MPN/100 mL	$[1/1 + K_T(t/x)]^x$	$[1/1 + K_T(t/x)]^x$
K_{20}, d^{-1}	2.6	2.6
θ	1.19	1.19
t,d	HRT in the system	HRT in the system
x	# of wetland cells in series	# of wetland cells in series
Background concentration (cfu/100 mL)	2,000	2,000
Note: this model was developed for facultative ponds and is believed to give a conservative estimate for fecal coliform removal in both FWS and SF wetlands.		

* MPN = Most Probable Number

10.8.7 Vector Control

Free water surface systems provide ideal breeding habitat for mosquitoes. Plans for biological control of mosquitoes through the use of fish that prey on mosquito larvae, fish and swallows should be incorporated in the design. Thinning of vegetation may also be necessary to eliminate pockets of water that are inaccessible to fish.

Mosquito breeding should not be a problem in SF systems, provided the system is designed to prevent mosquito access to the subsurface water zone. The surface is normally covered with pea gravel or coarse sand to achieve this purpose.

10.8.8 Vegetation Harvesting

Harvesting of the emergent vegetation is only required to maintain hydraulic capacity, promote active growth, and avoid mosquito population growth. Harvesting for nutrient removal is not practical and is not recommended.

10.8.9 Monitoring

Monitoring is necessary to maintain loadings within design limits. A routine monitoring program should be established for the following parameters:

a. Wastewater application rates (m^3/m^2 d).
b. Discharge flowrates (m^3/d).
c. Wastewater quality, including BOD_5 and COD, SS, Total Dissolved Solids (TDS), TN, total phosphorous, pH, and Sodium Adsorption Ratio (SAR).
d. Discharge water quality according to the analyses summarized in item c.

10.9 References

Canadian Council of the Ministers of the Environment. Technical Supplement 3. *Canada-wide Strategy for the Management of Municipal Wastewater Effluent. Standard Method and Contracting Provisions for the Environmental Risk Assessment.* Canadian Council of the Ministers of the Environment, June 2008. <https://ccme.ca/en/res/mwwe_techsupp_3_e.pdf>.

Crites, R.W., *Design Criteria and Practice for Constructed Wetlands*, Water Science and Technology, Vol. 29, No. 4, 1994.

Environment and Climate Change Canada. *Treatment processes for removal of ammonia from municipal wastewater*. Government of Canada, 2003.

Ontario Ministry of the Environment,. *Design Guidelines for Sewage Works. Government of Ontario*. Government of Ontario, 2019. <https://www.ontario.ca/document/design-guidelines-sewage-works-0>.

Reed, S.C., Crities, R.W., and Middlebrooks, E.J., *Natural Systems for Waste Management and Treatment*, McGraw-Hill Professional, 1995.

Stahre, P., and B. Urbonas, *Stormwater Detention for Drainage, Water Quality, and CSO Management*, Prentice Hall, 1990.

Tchobanoglous, George, Burton, Franklin L., Stensel, H. David, and Metcalf & Eddy. *Wastewater Engineering: Treatment and Reuse*. 4th Ed. / Revised by George Tchobanoglous, Franklin L. Burton, H. David Stensel. ed. McGraw-Hill Series in Civil and Environmental Engineering. Boston; Montreal: McGraw-Hill, 2003.

U.S. Environmental Protection Agency, *Design Manual - Constructed Wetlands and Aquatic Plant Systems for Municipal Wastewater Treatment*, September 1988.

U.S. Environmental Protection Agency. *Design Manual: Phosphorus Removal*. USEPA, 1987. <https://cfpub.epa.gov/si/si_public_record_report.cfm?Lab=ORD&dirEntryId=43610>.

U.S. Environmental Protection Agency. *Process Design Manual for Nitrogen Control*. USEPA, 1975. <https://cfpub.epa.gov/si/si_public_record_Report.cfm?Lab=NRMRL&dirEntryID=34313>.

Wastewater Committee of the Great Lakes - Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers. 2014. *Recommended Standards for Wastewater Facilities*. Health Research, Inc., Health Education Services Division. <https://www.health.state.mn.us/communities/environment/water/docs/tenstates/tenstatestan2014.pdf>.

Water Environment Federation, *Manual of Practice FD-12, Alternative Sewer Systems*, 1986.

Water Environment Federation, *Manual of Practice FD-16, Natural Systems for Wastewater Treatment*, 1990.

Water Environment Federation, Manual of Practice FD-16, Natural Systems for Wastewater Treatment, 2001.

Water Environment Federation. Nutrient Control WEF *Manual of Practice FD-7. Water Environment Federation*, Washington 1983.

Table of Contents

Chapter 11 Treated Effluent Disposal to Land

11.1 General

With many communities throughout the world approaching or reaching the limits of their available water supplies, reclaimed water use has become an attractive option for conserving and extending available water supplies. Reclaimed water use is the controlled application of treated wastewater by irrigation (onto the land surface to achieve disposal, utilization, and/or treatment of the wastewater) or by infiltration (into the soil). This can be achieved by a number of options including land application, surface and subsurface irrigation, in-ground trenches, and overland flow, as approved by Regulatory Authorities having jurisdiction.

Water reclamation and non-potable reuse only require conventional water and wastewater treatment technology that is widely practised and readily available in countries throughout the world. Furthermore, because properly implemented non-potable reuse does not entail significant health risks, it has generally been accepted and endorsed by the public in the urban and agricultural areas where it has been introduced. This section provides information on planning considerations re-use applications, water quality considerations, and guidelines for wastewater irrigation and other re-use criteria.

In addition to the general Pre-design Report requirements, the designer should include supplemental information as outlined in Section 1.3.

11.1.1 Definitions

Chemical Oxygen Demand
A quantitative measure of the amount of oxygen required for the chemical oxidation of carbonaceous (organic) material in wastewater using inorganic dichromate or permanganate salts as oxidants in a 2-hour test.

Infiltration
The flow or movement of water through interstices or pores of soil or other porous medium.

Irrigation
The artificial application of water to lands to meet the water needs of growing plants not met by rainfall.

11.2 Treated Effluent Application Methods

11.2.1 General

Land application of treated wastewater effluent is a method of disposing of effluent without direct discharge to surface waters. Ground disposal installations are normally used where the waste contains pollutants which can successfully be removed through distribution to the soil mantle. These pollutants can be removed through organic decomposition in the vegetation-soil complex and by adsorptive, physical, and chemical reactions with earth materials. Preliminary considerations of a site for ground disposal should include the compatibility of the waste with the organic and earth materials, as well as the percolation rates and exchange capacity of the soils. The ground disposal of treated effluent will eventually recharge the local groundwater, therefore, the quality, direction and rate of movement, and local use of the groundwater, present and potential, are prime considerations in evaluating a proposed site. Furthermore, the nutrient loads should also be of consideration as well.

It is essential to provide good vegetation growth conditions and removal of nutrients. It must be realized that a groundwater mound will develop below the application area after it is in use. The major factors in design of ground disposal fields are topography, soils, geology, hydrology, weather, agricultural practice, adjacent land use, equipment selection, and installation. The impacts of climate change should be considered in design (e.g., potential changes to groundwater elevations and avoidance of low-lying and coastal areas prone to flooding). Refer to Chapter 2 for further guidance.

The primary methods used for distributing treated effluent on the land are irrigation and infiltration.

Table 11.1 outlines various features and performance of treated effluent land application systems.

Table 11.1: Comparison of Features & Performance for Treated Effluent Utilization, Treatment, & Disposal Systems

System Requirement	Standard Rate Irrigation	Rapid Infiltration (RI)
Soil permeability	Moderate (medium texture soil)	Rapid (loamy sands and gravels)
Utilization of water and nutrients	High	None
Slope	Up to 30% for sprinkler and 6% for surface methods	Not critical
Storage	High (7 to 9 months)	N/A
Land area	High	Low
Water quality (e.g., salinity)	Very high	Medium to low
Treatment efficiency	Very high	Medium
Loading rate	500 – 6,000 L/m² a	6,000 – 100,000 L/m² a

11.2.2 Irrigation

11.2.2.1 Piping to Sprinklers

The piping should be arranged to allow the irrigation pattern to be varied easily. Stationary systems are preferred but if a moveable system is proposed, one main header must be provided with individual connections for each field and sufficient spare equipment must be available to assure non-interrupted irrigation. Facilities must be provided to allow the pipes to be completely drained at suitable points to prevent freezing and spillage of treated effluent into sensitive areas.

11.2.2.2 Sprinkling System

Sprinklers should be located to give a non-irrigated buffer zone around the irrigated area, and design of the buffer zone should consider wind transport of the treated effluent. The system should be designed to provide an even distribution over the entire field.

The selected application rate should be low enough to allow the irrigated treated effluent to percolate into the soil and to assure proper residency within the soil mantle. Proposed application rates will not be accepted without substantiating data.

In general, sufficient monitoring controls should be provided to indicate the degree of efficiency with which the sprinklers are working. A pressure gauge and flow meter should be provided.

11.2.2.3 Site Buffer Zone

The requirements for buffer zones around the irrigation operation are outlined in Section 11.3.3.2, and are dependent on a number of site-specific factors.

11.2.3 Rapid Infiltration

11.2.3.1 Applicability

Rapid infiltration involves the application of treated effluent to land by means of basins. The treated effluent percolates through the soil, undergoes a variety of physical, chemical, and biological reactions and eventually reaches the groundwater. The loss of water via plants or evaporation is minor compared to the loss by percolation. The loading must be intermittent to allow for the restoration of aerobic conditions in the soil. Acceptable salinity, boron, nitrogen, and phosphorus levels in the treated effluent will be governed by the potential use of the groundwater downstream of the RI site. The permeability of the site is, however, very important to the performance of a RI system, therefore, the SAR of the effluent should be below 9.

Optimum site conditions for RI are dependent upon the quantity of wastewater to be treated and the degree of treatment required. Generally, there will be an inverse relationship between maximum wastewater application rate and the degree of treatment. Soil conditions required for a good RI site are a deep uniform sandy loam to loamy sand having the chemical characteristics listed in Table 11.2.

Table 11.2: Optimal Chemical Characteristics of Soil Used for Rapid Infiltration Applications

Characteristic	Amount
pH	6.0 - 8.5
Organic matter	0.5 - 3.0%
Electrical Conductivity (EC)	2 dS/m
SAR	10
Cation exchange capacity	10 meq/100 g
Free Ca or Mg CO_3 should be present	

Rapid infiltration installations require permeable granular subsurface materials. A minimum of 4.0 m separation between the water table and the basin bottom between irrigation cycles is recommended. In situations where, potable water systems will not be affected and tertiary treatment is provided, the 4.0 m vertical separation distance may be reduced. As a minimum, the separation distance should be 1.0 m between the water table and the bottom of the basin during operation. Adverse natural groundwater conditions can be modified by the installation of underdrains and/or recovery wells.

Excessive slopes will restrict the usefulness of a RI site. The maximum slope is that which maintains downward infiltration with no premature lateral discharge. Generally, the maximum slope is 5% unless considerable earth moving is undertaken. Uniform flat topography will reduce construction costs. In areas where facultative lagoons are used for treatment, the lagoons will generally be large enough to provide cold weather storage, however, the infiltration area will have to be large enough to treat the annual wastewater production during the warm weather period. Treated effluent from treatment plants with short detention times will retain sufficient heat to allow continuous RI treatment and eliminate the need for storage.

11.2.3.2 Area & Infiltration Rate

Prior to site selection, the designer must determine the approximate land area required for an RI system. This can be obtained by using wastewater flow data and the annual amount of infiltration per unit area. The hydraulic conductivity required to estimate total infiltration can be determined from Table 11.3 and the following calculations. It is then suggested that a factor of 1.5 be applied to the calculated area requirements.

Table 11.3: Hydraulic Conductivities of Various Granular Deposits

Deposit	Hydraulic Conductivity (cm/s)
Clean and well sorted sand and gravel.	10^{-1}
Clean sand and moderately sorted gravel.	10^{-2}
Moderately sorted sand and gravel.	10^{-3}
Poorly sorted sand and gravel.	10^{-4}

Infiltration capacity is estimated by the following procedure:

- Estimate site hydraulic conductivity in (cm/s).
- Determine annual hydraulic loading and convert L/m² d to m/a (multiply by 0.365).
- Interpolate the site area (on the y-coordinate of Figure 11.1) using the line most closely representing the estimated HLR determined. The site area can also be determined by dividing the annual average treated effluent flowrate by the design annual hydraulic loading as shown below.

If seasonal treated effluent flows are not equalized, the highest average seasonal flowrate should be used for design. The initial estimate of required land area computed using the equation above may be adjusted depending on constraints, as discussed in the section dealing with the layout of the infiltration area.

- Maximum daily infiltration capacity of the site in question can be read off the x-coordinate (Figure 11.1).

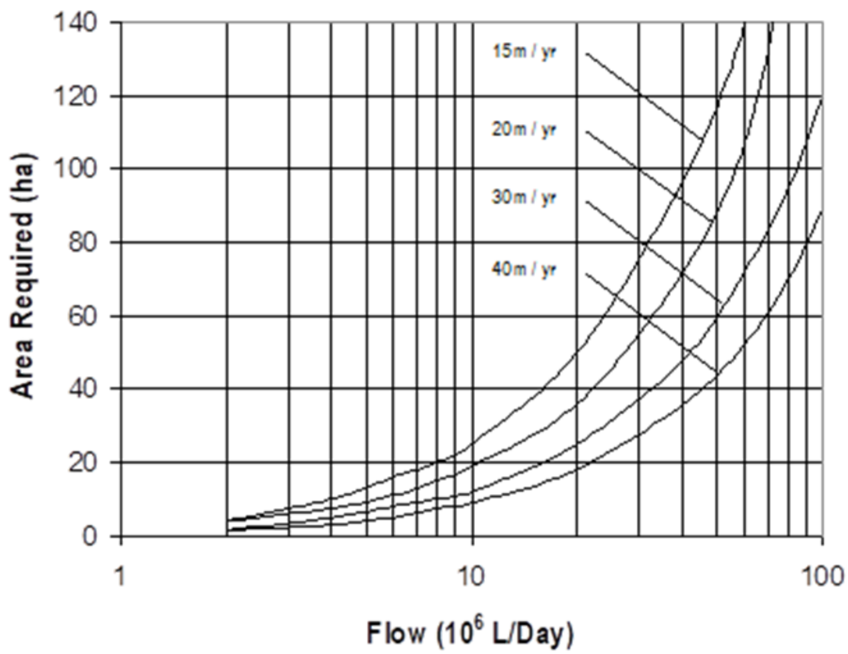


Figure 11.1: Determination of Land Area Required for Rapid Infiltration Systems

The infiltration rate must be confirmed by field testing. Surface area requirements for an RI system must include:

- Infiltration basins and dykes.
- Maintenance and laboratory building(s).
- Possibly on-site treatment facilities.
- On-site roads.
- Expansion and emergency use areas.
- Buffer strips.

11.2.3.3 Loading Cycle

In Atlantic Canada, RI systems would likely require an altered loading cycle with respect to seasons because longer resting periods may be required for soil drying and aeration during winter. Decreasing the application rate and increasing the length of the application and resting period are possible means of overcoming the problems of winter application.

Suggested loading cycles are shown in Table 11.4. The values given in this table are considered guidelines. Actual loading cycles should consider site-specific conditions.

Table 11.4: Suggested Hydraulic Loading Cycles for Rapid Infiltration Systems

Objective of Preapplication Treatment Period	Season	Application Period (Days)	Drying (Days)
Maximize infiltration rates of nitrification	Summer	1 - 3	4 - 5
	Winter	1 - 3	5 - 12
Maximize nitrogen removal	Summer	7 - 9	10 - 15
	Winter	9 - 12	12 - 18

11.2.3.4 Application Rate

Once the loading rate and loading cycle have been established, the application rate can be calculated. For example, if the HLR is 20 m/annum and the loading cycle is 1 day of application alternated with 7 days of drying, the application rate is as follows:

$$A = B \times \frac{(C + D)}{C} \times E$$

A = Daily application rate
B = Hydraulic loading rate
C = Time on
D = Time off
E = Conversion factor (annual to daily)

The application rate should be used to determine the maximum depth of the applied treated effluent. For instance, if the measured basin infiltration rate is 1.7×10^{-4} cm/s the maximum wastewater depth will be the daily application rate minus 1.7×10^{-4} cm/s.

In general, maximum treated effluent depth should not exceed 50 cm with a preferable maximum depth of 30 cm. If the treated effluent depth calculation indicates the recommended maximum will be exceeded, either the loading rate should be decreased, or the loading cycle adjusted until the maximum basin depth is acceptable.

11.2.3.5 Monitoring

A monitoring program should provide applied treated effluent quality, the quality of groundwater affected by the RI system and, if required, an analysis of the soil affected by the RI system. Several groundwater samples should be collected from sites expected to be influenced by RI and compared with samples from areas not affected by treated effluent infiltration.

11.2.3.6 Separation Distances

The requirements for RI separation distances are outlined in Section 11.3.3.2.

11.2.4 Runoff

The system should be designed to prevent surface runoff from entering or leaving the project site. Considerations for the impacts of climate change on storm water runoff (e.g., changes to the frequency and intensity of precipitation events) should be accounted for during design. Refer to Chapter 2 for further guidance.

11.2.5 Fencing & Warning Signs

The project area should be enclosed with a suitable fence to exclude livestock and discourage trespassing, depending on the level of treatment provided and type of effluent disposal used. A vehicle access gate of sufficient width to accommodate mowing equipment should be provided. All access gates should be provided with locks.

Appropriate signs should be provided along the fence around the project boundaries where necessary to designate the nature of the facility and advise against trespassing.

11.3 Guidelines for Treated Effluent Irrigation

Treated municipal effluent does not always meet a quality standard that would enable its unrestricted discharge to the sensitive receiving environment. For land application, concerns remain with respect to elevated concentrations of soluble salts, nutrients, and microbiological quality of the treated effluent.

The major difference between municipal treated effluent and “high quality irrigation water” is the higher concentration of living and nonliving organic material, nitrogen, phosphorus, and in some instances, higher sodium and salt levels in the municipal treated effluent. Low concentration of grease, oil, detergents, and certain metals may also be present, but these are generally at concentrations that do not adversely impact crops and/or the land if applied through irrigation at rates compatible with a crops seasonal water deficit need. Treated effluent suitability for irrigation is based on a select set of water quality parameters to be tested prior to and during their release. Site acceptability is to be based on pertinent soil and geological properties, topography, hydrology, as well as zoning and cropping intentions.

In contrast with natural irrigation waters, municipal treated effluent has numerous additional health and environmental factors that need to be evaluated to ensure no detrimental impacts occur from their use. Due to the origin, the variety and the often-changing quality of wastewater generated by municipalities, it is imperative that municipal treated effluent be tested for a much wider range of water quality parameters than is currently necessary for irrigation with natural waters. Irrigation with municipal treated effluent is a suitable disposal option in Atlantic Canada where additional moisture can be effectively utilized for improved crop production.

Treated effluent loading is to be based on the consumptive water use of the crop being grown. This loading, however, must also consider seasonal moisture deficiencies including climate change considerations (e.g., potential increases in the occurrence of drought conditions), system application efficiencies, and additional considerations related to annual soil leaching and crop nutrient utilization factors. The primary objective should be enhancement of crop production. The root zone of productive soils can often serve as one of the most active media for the decomposition, immobilization, or utilization of wastes. Considering these active processes in the topsoil, treated effluent can often be safely released to land at water quality standards less restrictive than those that would apply to a surface water release option. Further, with the added benefits currently applied to waste re-utilization processes and water conservation practices, treated effluent irrigation is considered an attractive waste disposal option.

11.3.1 Assessment of Municipal Effluent Quality for Treated Effluent Irrigation Development

As water quality standards for municipal treated effluent discharging to surface water bodies become more stringent, the associated treatment costs correspondingly escalate. Irrigation is, therefore, becoming a more desired alternative for treated effluent disposal for many communities, however, since different water quality variables need to be considered when evaluating WWTP effluents as a potential irrigation water source than those considered for its direct discharge into a receiving stream, a specific set of treated effluent quality reporting requirements must be outlined and defined. In this overall investigation it is, therefore, important to first evaluate restrictions that may apply to the use of standard sources of irrigation water and then consider what supplemental evaluations would apply to treated effluent irrigation use.

11.3.1.1 Natural Irrigation Water Quality Characterization

The use of waters for irrigation application normally involves evaluation of the following water quality parameters:

- Electrical conductivity is a reliable indicator of the TDS (salts) content of the water. The addition of irrigation water to soils adds to the concentration of salt in the soil. Concentration of these salts will result in an increase in osmotic potential in the soil solution interfering with extraction of water by the plants. Toxic effects may also result with an increase in salinity. Electrical conductivity is measured in dS m^{-1}. For specific values on acceptable EC levels in waters used for irrigation, refer to Table 11.5 that follows.
- Sodium adsorption ratio is an indicator of the sodium hazard of water. Excess sodium in relation to calcium and magnesium concentrations in soils destroys soil structure that reduces permeability of the soil to water and air. Sodium may be toxic to some crops.

$$SAR = \frac{NA^{+}}{\sqrt{\frac{Ca^{2+} + Ma^{2+}}{2}}}$$

(for concentrations in me/L)

and

$$SAR = \frac{NA^{+}}{\sqrt{Ca^{2+} + Mg^{2+}}}$$

(for concentrations in mmole/L)

Cations are expressed in mequivalent of charge/L or *mmoles of charge/L.*

For specific values on acceptable SAR levels in waters used for irrigation, refer to Table 11.5.

- Boron (B) is very toxic to most crops at very low levels. In most jurisdictions, excess natural boron in soils and water has not been a problem. Acceptable boron concentrations for agricultural use are included in the applicable sections of the most recently published Canadian Environmental Quality Guidelines (CEQG).
- Bicarbonate (HCO_3) is considered hazardous when concentrations are excessive in some areas and not in others. Waters of high bicarbonate concentrations have been used for many years with no adverse effects in some jurisdictions. Acceptable bicarbonate concentrations for agricultural use are included in the applicable sections of the most recently published CEQG.

For further information on any other chemical parameters that may impact irrigation suitability from natural water sources, reference should be made to the applicable sections of the most recently published CEQG.

In light of the preceding factors, only two parameters (SAR and EC) are normally of concern when irrigating with most available water sources in most jurisdictions. The limits for these parameters are as follows:

Table 11.5: Irrigation Water Quality Standards

Parameter	Safe	Possibly Safe	Hazardous
EC dS m^{-1}	< 1.0	1.0 - 2.5	> 2.5
SAR	< 4	4 - 9	> 9

The limits under the heading "Safe", are considered safe for all conditions. The "Possibly Safe" limits are considered safe for some conditions. Decisions should be based on the advice of a specialist. The "Hazardous" limits are considered unsuitable for almost all conditions.

Conditions to be assessed when dealing with waters that are "Possibly Safe" are as follows:

- **Climate of the area:** the deficit dictates the amount of water applied and consequently the amount of salt applied. Considerations for the impacts of climate change (e.g., potential increases in the occurrence of drought conditions) should be accounted for. Refer to Chapter 2 for further guidance.
- **Crops:** crops with high consumptive use require more irrigation water which again results in higher salt applications.
- **Irrigation practices:** frequent irrigation results in less leaching than less frequent water applications. Light, frequent irrigation results in more evaporation. Fall irrigation results in increased leaching.
- **Internal drainage:** good internal drainage facilitates rapid leaching of salts out of the root zone. System designs for irrigation with possibly safe water quality require specific investigation and the services of a specialist.

11.3.1.2 Comprehensive Treated Effluent Characterization

In contrast with fresh irrigation water, municipal treated effluent has additional health and environmental factors that need to be considered to ensure no detrimental impacts occur from its use. Due to the origin, variety and often changing quality of treated effluent generated by municipalities, towns, and private sources, it is imperative that municipal treated effluents be periodically tested for a much wider range of water quality parameters than is currently necessary for irrigation with fresh waters. A comprehensive characterization of the treated effluent is necessary as part of the initial treated effluent irrigation application process and subsequently as may be specified by the Regulatory Authority having jurisdiction. Annual monitoring of a number of key biological and chemical indicator parameters, both prior to and subsequent to any treated effluent irrigation, should also be performed. The comprehensive treated effluent quality characterization requirements and the annual treated effluent quality monitoring requirements are discussed further in subsequent sections that follow.

The comprehensive characterization of treated effluent quality provides a means to ensure a basic level of irrigation quality control. It also provides useful baseline information to evaluate impacts from future irrigation. These impacts may relate to changes that occur in community water sources, waste treatment processes, community size, and community or industrial discharge loadings. In addition, the treated effluent quality characterization process may also provide an opportunity for community planners and engineering consultants to better evaluate the effectiveness of the treatment process and its ability to eliminate harmful constituents that could normally restrict the potential for irrigation use. The requirement of a comprehensive testing analysis in the initial application may enable future analytical testing requirements to be less onerous while still ensuring adequate protection of human health and the environment.

11.3.1.2.1 General Health Related Aspects

Biological assessment of municipal treated effluent is obtained by means of biological counts performed on the treated effluent prior to or on release. Potential human pathogens of concern found in domestic wastewater may be grouped into the following four categories:

1. Bacteria (*Salmonella*, *Shigella*, *Mycobacterium*, *Klebsiella*, *Clostridium*).
2. Protozoan parasites (*Entamoeba*, *Giardia*, *Trichomonas*).
3. Helminth parasites (*Ascaris*, *Toxacara*, *Taenia*, *Trichuris*, *Enterobius*).
4. Viruses (*Picornaviruses*, *Adenoviruses*, *Rotaviruses*).

The types and numbers of pathogenic organisms in wastewater depend on the nature of the wastewater being treated and the type of wastewater treatment provided. Wastewater organisms such as bacteria and viruses that are adsorbed to particulate matter tend to co-precipitate during settling phases of wastewater treatment and are thereby partly removed as solids from the water phase (Moore et al. 1975). Similarly, encysted and egg stages of parasites, with specific gravities 1.06 to 1.20 (Englebrecht 1978), are effectively removed from the liquid wastewater during the settling phases of wastewater treatment process.

The use of trickling filters, activated sludge systems, and effluent disinfection are additional treatment processes traditionally used to further reduce certain pathogenic organisms in wastewater, however, there is no single wastewater treatment process which will remove all pathogenic microorganisms. Many potentially disease-causing microorganisms will, therefore, continue to exist in wastewater. The types and amounts of these microorganisms will vary greatly with the treatment process or combination of the processes utilized, therefore, for wastewater irrigation to be authorized, the minimum treatment requirement is secondary treatment followed by disinfection, and storage as required by the Regulatory Authorities having jurisdiction. Despite their presence, the potential health hazard associated with utilizing treated effluent for irrigation can be minimized by adopting certain precautions and procedures.

The majority of the potentially harmful microorganisms are killed over a period of time by exposure to strong sunlight, high temperatures, and dry weather that may allow their direct application for sites with restricted access. Disinfection of treated effluent prior to land application should be required where warranted by public health concerns (e.g., golf courses and parks). Bacteriological quality should meet the standards outlined in Table 11.7.

The timing of effluent irrigation with respect to harvesting crops and grazing livestock is also a factor that must be addressed. See Section 11.3.3.4.

Assessment of bacteriological constituents for the comprehensive treated effluent characterization requires only the testing of *E. coli* and/or fecal coliforms. Additional testing for other bacteriological parameters has not been found to be necessary in some jurisdictions as adoption of a best practical treatment approach requiring primary treatment, storage, and various crop restrictions before irrigation, has proven appropriate in protecting the public from any adverse exposures to these particular constituents.

11.3.1.2.2 Other Water Quality Aspects

Other water quality aspects to be included in the comprehensive treated effluent characterization assessment prior to the development of a treated effluent irrigation system are included in the following sections.

General Chemical Parameters

The general parameters are those that are analyzed to assess the effectiveness of the wastewater treatment process and to evaluate variability in the quality of the wastewater prior to its release to the environment. They also represent water quality values that, if exceeded, can often restrict treated wastewater sources from being considered for irrigation purposes.

- Biochemical oxygen demand typically below 25 mg/L for most municipal treated effluents following secondary treatment.
- Total suspended solids typically below 25 mg/L for most municipal treated effluent following secondary treatment.
- Chemical oxygen demand typically below 50 mg/L for most municipal treated effluents following secondary treatment.
- pH typically ranges from 6.5 to 8.5 for most municipal treated effluents. These values are comparable to most natural surface waters and are considered to pose no restriction to irrigation use. A continued long-term use of waters outside this pH range could eventually alter naturally occurring pH levels in surface soils to which they are applied and, therefore, could possibly lead to micronutrient imbalances and potential future crop production and fertility problems.
- Electrical conductivity values range widely within municipal treated effluent and like some natural water sources exceed levels that would be recommended for irrigation.

Those municipal treated effluent with EC values less than 1.0 dS/m are considered of good quality and should pose no problems for irrigation use, unless the SAR of the treated effluent is greater than 4.

Municipal treated effluents found to have EC values between 1.0 and 2.5 dS/m are considered marginal for irrigation and are usually restricted to use on land with favourable internal drainage properties. Crops normally grown under irrigation with such municipal treated effluent would not be impacted significantly. For situations where treated effluent of this quality is utilized for irrigation on a regular ongoing basis, supplemental approval conditions, requesting the periodic testing and reporting of salinity levels for lands being irrigated, would most likely apply. Results from such testing should be reported to the Regulatory Authority having jurisdiction, if:

- Complaints of adverse impacts to the irrigated lands have been raised.
- Or an application for approval renewal was being processed and concerns over deteriorating crop conditions were an issue.

Provision for periodic salt leaching is often advisable when considering treated effluent irrigation with water in this EC range.

Treated effluents with EC values exceeding 2.5 dS/m must not be used for irrigation purposes. Any such application would be restricted to a low volume discharge situation and require supplemental monitoring and reporting to be compiled on a regular basis.

It may be noted that EC values are often high in communities that utilize groundwater as a water supply source. Improving the quality of water supplies for these communities or adopting an alternate water supply source can lead to improvement in final treated effluent EC levels for these communities and possibly improve its suitability for irrigation.

Sodium adsorption ratio values can vary widely within municipal WWTPs and like many natural water sources can often occur at levels that restrict their use for irrigation applications. Since adverse effects from high SAR are also dependent on the associated EC levels of the treated effluent, one should be aware of this interrelationship when evaluating SAR.

As a general guide treated effluent having SAR values less than 4 pose no problem for irrigation use.

Municipal treated effluents with SAR values ranging between 4 and 9 are considered marginal for irrigation and must include careful management to avoid potential damage to the land base or reduced crop productivity. Applying treated effluent of this quality can be particularly damaging on very fine textured soils or in situations where EC values of the treated effluent are less than 1.0 dS/m. Occasional calcium nitrate or gypsum applications may be helpful as a supplemental management practice on lands receiving irrigation applications of this quality for long periods of time. For situations where marginal municipal treated effluent quality is utilized for irrigation, supplemental approval conditions, including periodic testing would apply. Results from such testing should be reported to the Regulatory Authority having jurisdiction if either:

- Complaints of adverse impacts to the irrigated lands have been raised.
- Or application for approval renewal was being processed and concerns of deteriorating soil quality or reduced crop productivity were an identified issue.

Treated effluent with SAR values exceeding 9 should not be used for irrigation.

Communities using ion-exchange process for water softening can significantly increase SAR values in the wastewater. Hence, careful and regular monitoring of SAR levels within systems where water softeners are used is important.

Nutrients

One of the main advantages of using treated effluent irrigation is that it may often enhance the fertility of the lands to which it is applied. This can add considerably to potential crop yield and, therefore, the associated agricultural resource value. Nutrient loading rates, while significant, are seldom at levels that would present a concern when using municipal treated effluent for irrigation. Most nutrient levels are well within the range that can be assimilated by plants if the treated effluent is applied at a rate and frequency that conforms to active crop growth. Potential contamination of groundwater would only be a concern under extremely shallow groundwater levels, unsuitable soil conditions, or gross mismanagement of the applied treated effluent. Since all these factors are carefully considered as part of these Guidelines, potential contamination of the groundwater should not present a concern. The following nutrients should be analyzed and reported as part of the comprehensive treated effluent quality characterization process:

1. Nitrogen can be evaluated in a number of different forms. Regular evaluation of nitrogen by analyzing for NO_3-N, NH_3-N, NO_2-N, and TKN should be conducted. The typical concentration for total nitrogen of most municipal treated effluent is up to 20 mg/L. This means that if 30 cm/year of treated effluent were applied, an N loading of 30 to 60 kg/ha/year would be applied to the land base. Providing treated effluent is not applied in quantities that exceed the field moisture capacity during periods of treated effluent applications, and is applied during the active crop growing season, such loadings can be easily assimilated by the growing crop without harmful health or environmental concerns developing. If nitrogen is applied at a rate that exceeds the ability of the plant and soil system to contain it or convert it to nitrogen gas, the excess nitrogen may pass through the surface soil and into groundwater. High concentrations of nitrogen, particularly in the form of nitrate, are considered a health hazard in drinking water. Treated effluent that consists of a total nitrogen concentration within the typical range can easily be assimilated by the growing crop without harmful health or environmental concerns provided treated effluent is not applied in quantities that exceed the field moisture capacity and it is applied during active crop growing season.
2. Phosphorus is to be evaluated as total phosphorus. The typical concentration of total phosphorus in municipal treated effluent following secondary treatment is up to 6 mg/L. If 30 cm/year of treated effluent were applied, this would translate to a phosphorous loading of 6 to 18 kg/ha/year. Since these levels are considered to be reasonably low and phosphorus is effectively immobilized in most soils at shallow depths, the potential for adverse impacts on groundwater quality is remote. Care must be exercised, however, to ensure treated effluent applications are applied at rates that do not exceed the infiltration capacity of the soils as high phosphorus levels in surface runoff and erosion sediments can create significant environmental concern if washed into neighbouring lakes, streams, or other surface water bodies.
3. Potassium is another major nutrient present in treated effluent of value for crop production that should be evaluated. The typical concentration for potassium in most municipal treated effluent is up to 40 mg/L. If 30 cm/year of treated effluent were applied this would translate to a K loading of 15 to 120 kg/ha/year. Such levels are normally assimilated by crops and are thus not considered to be an environmental or health risk.

Major Cations and Anions

The treated effluent should be analyzed and reported for the following cations and anions in the required comprehensive treated effluent characterization:

- Calcium (Ca) mg/L.
- Sodium (Na) mg/L.
- Bicarbonate (HCO_3) mg/L.
- Fluoride (F) mg/L.
- Chloride (Cl) mg/L.
- Magnesium (mg) mg/L.
- Carbonate (CO_3) mg/L.
- Alkalinity, total ($CaCO_3$) mg/L.
- Sulphate (SO_4) mg/L.

Metals

Uptake of harmful amounts of toxic heavy metals by plants is not considered a potential risk in use of municipal treated effluent, as most metals are removed from the wastewater in the primary treatment process, however, as a precautionary measure, all wastewater should be initially tested for the following metals in Table 11.6 levels are below recommended CCME, CEQG, prior to granting authorization for irrigation application.

Since collection of this information is intended more as a general treated effluent quality characterization inventory rather than for purposes of assessing irrigation water quality limits, specific values will likely not be exceeded for most municipal treated effluent tested.

In addition, a careful evaluation of any industrial discharges into the municipal system and their potential impact on overall wastewater quality must also be addressed. If due to the nature of these industrial activities, concerns relating to any other chemicals become evident, then these chemicals should also be added to the comprehensive list of suggested chemical parameters for treated effluent characterization.

Table 11.6: Canadian Environmental Quality Guidelines for the Protection of Agricultural Water Uses

Parameter	Concentration (μg/L)[a]	Remarks[b]
Aluminum	5,000	Can cause non-productively in acid solids (pH < 5.5), but more alkaline solids at pH > 5.5 will precipitate the ion and eliminate any toxicity.
Arsenic	100	Toxicity to plants varies widely, ranging from 12 mg/L for sudangrass to less than 0.05 mg/L for rice.
Beryllium	100	Toxicity to plants varies widely, ranging from 5 mg/L for kale to 0.5 mg/L from bush beans.
Boron	500 - 6,000	Boron is very toxic to most crops at very low levels. In most jurisdictions, excess natural boron in soils and water has not been a problem.
Cadmium	5.1	Toxic to beans, beets, and turnips at concentrations as low as 0.1 mg/L in nutrient solutions. Conservative limits recommended because of its potential for accumulation in plants and soils to concentrations that may be harmful to humans.
Chromium - Trivalent Cr (iii) - Hexavalent Cr (vi)	 4.9 8.0	Not generally recognized as an essential growth element. Conservative limits recommended because of lack of knowledge on toxicity to plants.
Cobalt	50	Toxic to tomato plants at 0.1 mg/L in nutrient solution. Tends to be inactivated by neutral and alkaline soils.

Parameter	Concentration (µg/L)[a]	Remarks[b]
Copper	200 - 1,000	Toxic to a number of plants at 0.1 to 1.0 mg/L in nutrient solutions.
Fluoride	1,000	Inactivated by neutral and alkaline soils.
Iron	5,000	Not toxic to plants in aerated soils but can contribute to soil acidification and loss of reduced availability of essential phosphorus and molybdenum. Overhead sprinkling may result in unsightly deposits on plants, equipment, and buildings.
Lead	200	Can inhibit plant cell growth at very high concentrations.
Lithium	2,500	Tolerated by most crops up to 5.0 mg/L, mobile in soil. Toxic to citrus at low levels (> 0.075 mg/L). Acts similar to boron.
Manganese	200	Toxic to a number of crops at a few tenths mg/L to a few mg/L, but usually only in acid soils.
Molybdenum	10 - 50	Not toxic to plants at normal concentrations in soil and water. Can be toxic to livestock if forage is grown in soils with high levels of available molybdenum.
Nickel	200	Toxic to a number of plants at 0.5 to 1.0 mg/L; reduced toxicity at neutral or alkaline pH.
Selenium	20-50	Toxic to plants at concentrations as low as 0.025 mg/L and toxic to livestock if forage is grown in soils with relatively high levels of added selenium. An essential element for animals but in very low concentrations.
Tin	-	Effectively excluded by plants; specific tolerance unknown.
Titanium	-	See remark for tin.
Tungsten	-	See remark for tin.
Uranium	10	
Vanadium	100	Toxic to many plants at relatively low concentrations.
Zinc	1,000 - 5,000	Toxic to many plants at widely varying concentrations; reduced toxicity at pH > 6.0 and in fine-textured or organic soils.

* Limits are adopted from the *Canadian Environmental Quality Guidelines (CEQGs) Summary Table* from the Canadian Council of Ministers of the Environment (2005).

* Adopted from *Wastewater Engineering: Treatment and Resource Recovery* by Metcalf & Eddy (2003).

11.3.1.3 Annual Treated Effluent Quality Monitoring Requirements

Wastewater must also be analyzed, and results reported annually for certain water quality parameters, both prior to and on completion of each irrigation application event. This monitoring requirement is in addition to the comprehensive treated effluent characterization outlined in Section 11.3.1.2. For annual testing purposes the treated effluent should be sampled at the pipe inlet of the irrigation distribution equipment. The treated effluent quality for treated effluent irrigation should meet the standards specified in Table 11.7.

Table 11.7: Treated Effluent Quality Guidelines for Wastewater Irrigation

Parameter	Guidelines		Type of Sample	Comments
	Restricted Use	Unrestricted Use		
*E. coli**	< 200 MPN/100 mL	< 2 MPN/100 mL	Grab (twice/month)	For unrestricted use, sampling should be conducted prior to startup and on a weekly basis. For restricted use sampling should be conducted prior to startup and then one more sample sometime during discharge.
BOD	25 mg/L	10 mg/L	Grab/composite**	Sampling should be conducted at startup and once during discharge.
COD	50 mg/L	20 mg/L	Grab/composite**	Samples collected twice annually, prior to and on completion of a major irrigation event.
TSS	25 mg/L	10 mg/L	Grab/composite**	For unrestricted use, sampling should be conducted prior to startup and on a weekly basis. For restricted use sampling should be conducted prior to startup and then one more sample sometime during discharge.
EC	1.0 - 2.5 dS/m	< 1.0 dS/m	Grab/composite**	Samples collected twice annually, prior to and on completion of a major irrigation event.
SAR	4 - 9 for when EC >1.0 dS/m	< 4	Grab/composite** (once/month)	Samples collected twice annually, prior to and on completion of a major irrigation event.
pH	6.5 - 8.5		Grab/composite**	Samples collected twice annually, prior to and on completion of a major irrigation event.

* For golf courses and parks only.

** Grab sample would suffice if storage is provided; composite sample is required if storage is not provided.

11.3.1.4 Other Requirements

Table 11.7 provides guidelines for interpretations of water quality for irrigation.

Table 11.8: Guidelines for Interpretations of Water Quality for Irrigation

Potential Irrigation Problem		Units	Degree of Restriction on Use		
			None	Slight to moderate	Severe
Salinity (affects crop water availability)					
EC_w		dS/m or mmho/cm	< 0.7	0.7 - 3.0	> 3.0
TDS		mg/L	< 450	450 - 2,000	> 2,000
Permeability (affects irrigation rate of water into the soil. Evaluate using ECw and SAR or adj RNa together)[b]					
SAR or adj RNa = 0 - 3	and EC_w		≥ 0.7	0.7 - 0.2	< 0.2
SAR or adj RNa = 3 - 6	and EC_w		≥ 1.2	1.2 - 0.3	< 0.3
SAR or adj RNa = 6 - 12	and EC_w		≥ 1.9	1.9 - 0.5	< 0.5
SAR or adj RNa = 12 - 20	and EC_w		≥ 2.9	2.9 - 1.3	< 1.3
SAR or adj RNa = 20 - 40	and EC_w		≥ 5.0	5.0 - 2.9	< 2.9
Specific ion toxicity (affects sensitive crops):					
Sodium (Na)					
Surface irrigation		SAR	< 3.0	3.0 - 9.0	> 9.0
Sprinkler Irrigation		mg/L	< 70	> 70	
Chloride (Cl)					
Surface irrigation		mg/L	< 140	140 - 350	> 350
Sprinkler irrigation		mg/L	< 100	> 100	
Boron (B)		mg/L	< 0.7	0.7 - 3.0	> 3.0
Trace elements (see Table 11.6)					
Miscellaneous effects (affects susceptible crops)					

a. From *Wastewater Engineering: Treatment and Resource Recovery* by Metcalf & Eddy (2003).

b. For treated effluent irrigation, it is recommended that SAR be adjusted to include a more correct estimate of calcium in the soil water.

c. Overhead sprinkling only.

11.3.2 Assessment of Land Suitability for Proposed Treated Effluent Irrigation Development

Land classification and other relevant soil, climate, climate change considerations, and groundwater assessment activities are generally performed after completing the comprehensive treated effluent characterization assessment, and results of the treated effluent characterization have shown that the wastewater is suitable for irrigation.

Careful assessment and characterization of the land base including associated soil, groundwater, and other crop related inputs are required prior to proceeding with actual design of the treated effluent irrigation system. A site is classed as suitable for treated effluent application only if it is found to possess soil, climatic, and physical characteristics that enable effective utilization of the treated effluent applied without causing future damage to the land base or to the underlying groundwater. Site conditions must also be such that they effectively restrict any detrimental offsite movement of the treated effluent through leaching, groundwater migration, surface runoff, or drift from irrigation spray. The following sections outline land classification, soil, and other testing requirements that must be addressed prior to actual development of an applicable treated effluent irrigation system design and issuance of the authorized approval.

11.3.2.1 Site Suitability

Before treated effluent irrigation development can proceed, the lands to be irrigated must first be reviewed and approved by the Regulatory Authority having jurisdiction.

For purposes of the regulatory review, the following information should be provided:

- A map showing the location of all soil sampling, description of sites, and surrounding activities or uses.
- A copy of all soil logs.
- A copy of soil chemical and physical analysis completed for the classification.
- A legible soil map that shows the soil description for the affected areas.
- A drafted land classification map at a scale of 1-to-5,000 showing the land class symbol, drainability, and limitations for each unit classified.
- A remark sheet or report that accompanies the land classification map. The typed report should briefly describe each land class unit with regard to the type of soils, soil texture, irrigation suitability, suitability for gravity or sprinkler irrigation development, the limitations of the irrigable units and reasons why non-irrigable units are rated non-irrigable. A statistical summary table that shows the following, where applicable, should also be included:
  - Total irrigable acres.
  - Total non-irrigable acres.
  - Right-of-way and easement acres.
  - Not investigated acres.
  - Acres of farmsteads or other physical features that are present.

Municipal treated effluent has much higher nitrate levels than other irrigation water sources. It is, therefore, necessary to further restrict treated effluent application on lands where the natural water table is less than 1.0 m below ground surface and/or impermeable bedrock or other geological barriers exist at less than 1.0 m below ground surface.

The following soil and site characterization details must also be collected and reported, in addition to completing the required land classification designations and mapping.

11.3.2.1.1 Soil Assessment

Soil assessment involves examination of test pits and testing of soil permeability.

Test Pits

Test pits provide information about the soil profile at the proposed location of the irrigation system. This information must include the following:

- Organic layer.
- Total soil depth.
- Effective soil depth.
- Total depth of test pit.
- Root penetration.
- Depth to bedrock.
- Depth to layer of soil with unacceptable permeability.
- Determination of highest seasonal water table.
  - Presence and depth of mottling.
  - Depth to water.

  - Moisture content (saturated, moist, dry, etc.).
  - Perched water table.
- Soil profile:
  - Description of soil (including all soil from unacceptably high to unacceptably low).
  - Depth of each layer.
  - Texture of soil.
  - Moisture content (saturated moist, dry, etc.).
  - Density (loose, medium, compact, tight, etc.).
  - Colour.
  - Structure.

For safety, the pit should be more than 1.2 m deep, with sloping sides and an entrance ramp for easy access and escape in the event of a soil slide. All soil removed from the pit should be placed a minimum of 1.0 m from the edge of the pit. If the pit is dug by backhoe and verification of subsoil conditions is required, the pit may be taken to a greater depth, but inspection should be carried out from the surface with the aid of samples of soil recovered by the machine bucket. A soil profile can then be recorded based on the variation in soil characteristics with depth.

All test pits must be dug in compliance with the Regulatory Authority having jurisdiction.

In-situ Permeability Tests

In-situ permeability tests can be used to confirm the estimation of soil permeability based on the visual assessment of soil properties in the test pit. When using these tests to verify results, a minimum of three tests should be done. If the tests are not of similar order of magnitude, more tests should be conducted.

These tests may also be used for the:

- Determination of particular sandy gravel as a soil with acceptable or unacceptably high permeability.
- Determination of a particular soil as an unacceptably low or an impermeable soil.
- Confirmation of visual assessment of soils for higher flow systems, such as commercial and institutional buildings.

11.3.2.1.2 Soil Properties

Some soil properties that are useful in assessing soil suitability include:

- Texture.
- Structure.
- Colour.
- Density.
- Depth.

A soils consulting engineer should assess the soil results and make recommendations.

11.3.2.1.3 Topography

Topographic features such as relief, site and shape of fields, soil type and texture, brush/tree cover, and surface drainage features must be evaluated for site suitability.

Land may not be considered suitable for irrigation due to one or a combination of factors such as:

- Steep slopes.
- Hummocky relief.
- Brush/tree cover.
- Small or irregular shape.
- Sloughs.
- Wetlands.
- Rough broken topography.

The topography is to be classified as to its suitability for treated effluent irrigation. The topography at each site is also to be mapped. This topographic mapping should be provided at a level of detail not less than a scale of 1-to-10,000 and a contour interval of 0.5 m. The information should be gathered either from a topographic survey of the land parcel or from a suitable scaled orthophoto or photogrammetric mapping of the property. This mapping must reference grid and property boundaries, treated effluent irrigation development boundaries, and soil test and groundwater test site locations. Inclusion of recent stereoscopic air photo coverage at a 1-to-10,000 scale would be advisable but is not a requirement.

11.3.2.2 Other Requirements

Other information in the initial site assessment process must include:

- Location and mapping of any surface water courses, water bodies, or domestic wells located on or within 150 m of the treated effluent development site.
- Location and mapping of any residential dwelling on or within 400 m of irrigation sites and 150 m of infiltration sites.
- Location and mapping of all public roads, highways, or other public corridors on or within 30 m of the treated effluent development site.

These site-specific requirements are intended to provide baseline information on all sites to be developed for treated effluent irrigation purposes. The knowledge is intended to assist in evaluating potential impacts of long-term treated effluent irrigation on the land base over time.

11.3.3 Assessment of System Design Needs for Proposed Treated Effluent Irrigation Development

Treated effluent irrigation system design is undertaken once water quality assessment and land suitability assessment are affirmed. The design integrates treated effluent quality with land base limitations and restrictions that relate to cropping, climate including climate change considerations, application, and public acceptance issues. The overall design includes an account of the following items, which are further defined in the following sections:

- Climate.
- Land area.
- Application loading rates.
- Crop considerations.
- Treated effluent storage ponds.

11.3.3.1 Climate

There are a number of climate factors that must be considered to ensure an effective treated effluent irrigation system design. Considerations for how climate change may impact these factors should be included the assessment. The climate factors are defined as follows:

- Adequate storage must be provided for periods when treated effluent cannot be disposed of by irrigation due to unauthorized periods, climatic condition, wind speeds in excess of 30 km/hour, or during periods of intense or prolonged precipitation.
- Seasonal mean precipitation, evapotranspiration and seasonal crop moisture demands must, therefore, be established for the infiltration or for the irrigation period authorized and be applicable to the geographical area of the specific project. These requirements will be necessary to determine the land base required to effectively dispose of the annual volumes of community wastewater available for discharge. Sufficient land to handle this anticipated flow must be obtained. Irrigation systems should be designed to have an almost complete utilization of nutrients and about 85% utilization of water. Since annual values will vary from year to year, design must allow for either a 25% treated effluent storage carry over or provision for an occasional expansion in irrigation system and land base design in order to accommodate the lower treated effluent irrigation discharge allotments required during wet years. Provision for supplemental irrigation sources in dry periods may also be considered.

11.3.3.2 Land Area

There are a number of land-related factors relevant to irrigation system design that must be considered. These factors are defined as follows:

- Specific irrigation design features must be provided that will avoid application of irrigation treated effluent to any non-irrigable land areas (greater than 15% of the area to be irrigated).
- The amount of land and equipment required will depend upon the mean annual consumptive use of water by plants, natural precipitation from April through September, an irrigation efficiency factor, and an appropriate leaching requirement. The impacts of climate change should be considered in design (e.g., potential shifts in precipitation seasonality and potential increases in the frequency and intensity of precipitation events). If no provisions are provided for extra treated effluent storage during abnormally wet years, additional land areas and equipment will be required to meet these needs.
- The land area to be accumulated must also allow for any buffer zones or setback limits that apply on or around land areas where treated effluent irrigation is to be undertaken. Setbacks and buffer zones that apply are outlined in Table 11.9.

Table 11.9: Setback Requirements

Parameter	Requirements
Adjacent properties	Buffer zone of 15 m between irrigated land and adjacent property owners.*
Adjacent dwellings	Buffer zone of a minimum of 50 m and preferred set back of 100 m between irrigated land and any occupied dwellings.*
Public rights of way	Buffer zone of a minimum of 25 m between irrigated land and any public right of way.
Potable water wells	Buffer zone of a minimum of 30 m between irrigated land and any potable water well.
Watercourses, rivers, streams, etc.	Buffer zone of a minimum of 20 m and preferred set back of 50 m between irrigated land and any watercourse.**

* Distance may be reduced with the signed permission of adjacent property owner.

** Watercourses used for golf course irrigation area exempt from the buffer zone. Distance maybe reduced depending on the actual quality of irrigation water.

In addition to the above consideration, the land area to be used for treated effluent irrigation and storage cells should be sufficiently large such that treated effluent discharge will not occur during the following periods.

- Outside the growing season except if authorized for a fall irrigation.
- During and for 30 days prior to the harvesting of crops.
- During and for 30 days prior to grazing by dairy cattle.
- During and for 7 days prior to pasturing by livestock other than dairy cattle.

A plan illustrating the layout of the irrigation system designed to irrigate the site should be provided. The plan must illustrate:

- The boundaries of the particular section(s) within which irrigation application will take place.
- The boundaries of the land area to which treated effluent will be applied.
- The extra land area to be irrigated during wet seasons when above average mean seasonal precipitation or extreme precipitation occurs.
- If design for extra lagoon storage is not provided.
- The actual orientation of irrigation equipment.
- Sprinkler head sizing.
- Operating pressures.
- Overall irrigation system layout.

11.3.3.3 Application Loading Rates

The rate of treated effluent application loading should depend on individual crop moisture and nutrient uptake needs. These factors are defined as follows:

- Nitrogen is usually the only nutrient that may prove to be restricting in respect to the amount of treated effluent that may be applied in a given irrigation season. The amount of plant available nitrogen, based on amount of treated effluent that is applied, should be calculated and noted as kg per ha per year. As long as these rates do not exceed the annual crop nitrogen removal rates and an active crop-harvesting program exists no restrictions to the application of typical treated effluent should apply. Other major nutrients generally do not exceed annual crop uptake requirements and, therefore, do not pose a risk to water quality. If nitrogen is applied at a rate that exceeds the ability of the plant and soil system to contain it or convert it to nitrogen gas, the excess nitrogen may pass through the surface soil and into groundwater. High concentrations of nitrogen, particularly in the form of nitrate, are considered a health hazard in drinking water.

- Crop moisture requirements thus become the main determining factor in establishing acceptable treated effluent irrigation application limits. Annual treated effluent application amounts ultimately depend on the annual seasonal crop needs minus season rainfall, including climate change considerations (e.g., potential changes in seasonal precipitation). Still, other factors such as soil moisture holding capacity, soil infiltration rate, crop rooting depth, rate frequency and duration of irrigation event, irrigation system efficiency, and soil leaching requirements will have a bearing on the efficiency of crop moisture utilization and, therefore, need to be evaluated as part of any irrigation system design. A local irrigation specialist or a qualified agricultural consulting firm should be consulted to ensure accurate assessments of these values for different locations. The eventual design of the irrigation system must ensure effective uniform application of the treated effluent and prevent any surface runoff or prolonged surface ponding to occur during application. The irrigation system must also be designed to avoid treated effluent applications that exceed crop seasonal water deficit requirements and leaching demands. If natural precipitation during the irrigation off-season is not sufficient to enable leaching of excess salt accumulations, the irrigation system must account for an annual leaching factor of 10%. This being required to assist with flushing excess soluble salts below the crop root zone.

11.3.3.4 Crop Considerations

Only certain crops are deemed suitable for production on lands to be irrigated with treated effluent. Crops for direct human consumption are not suitable for effluent irrigation or infiltration. The current authorized crops include only forages, coarse grains, turf, trees, and oil seeds. See Section 11.3.1.4 for other considerations.

11.3.3.5 Treated Effluent Storage Ponds

The design of any storage reservoir required to retain treated effluent during periods of restricted irrigation must meet current design criteria as described in Chapter 8. If the pond is designed to hold treated effluent, then it is not required to have an impermeable liner. Where odour problems may occur, aeration of the storage reservoir may be necessary.

11.3.4 System Operation

Once the wastewater development project has been approved and constructed, an Approval/License/Permit to Operate is required before operation can proceed. The Approval/License/Permit to Operate will spell out operating conditions and requirements for the system.

The municipality must be responsible for the proper operation of the irrigation project, even if someone other than the municipality is managing the system. Proper operation of the system is essential for longevity of the system, for a high degree of treatment and for high production. Although crop production is not the prime objective of the system, a vigorous crop growth is essential for utilization of water and nutrients.

Due to the great variation in waste concentration, soils, and climate, no attempt will be made to elaborate further on irrigation management. Specific operational requirements will be stated in the Approval/License/Permit to Operate.

Operating conditions and requirements for the system must be described prior to receiving approval. Due to the great variation in waste concentration, soils, climate including climate change considerations, no attempt will be made to elaborate further on irrigation management in this document. Specific operational requirements will be stated in the certificate of approval.

11.4 Reuse of Treated Effluent for Golf Course Irrigation

Treated effluent for golf course irrigation, where acceptable to the Regulatory Authorities having jurisdiction, is treated to the extent that it can beneficially be reused without adverse effects of public health or the environment. Benefits of the use of treated effluent for golf course irrigation include:

- A more cost effective and environmentally beneficial alternative compared to other methods of treated effluent disposal.
- Conservation of water resources.
- Reduced demand on municipal water supply.
- Addition of nutrients and micronutrients is beneficial to turf growth.

Planning, design, and management of golf course irrigation systems that use treated effluent must consider the following:

- Regulatory concerns regarding protection of public health and the environment.
- Concerns about possible effects of treated effluent on golf course soils and vegetation.
- Cost associated with installation and operation of an irrigation system.

11.4.1 Environment

Positive environmental effects of irrigation with treated effluent include:

- Avoiding the need to discharge effluent into sensitive areas such as beaches or water supplies.
- Conservation of scarce water resources which are replaced by treated effluent.
- The nutrient content of treated effluent can provide an economic advantage by reducing the cost of commercial fertilizers.

Environmental concerns could include:

- Contamination of surface water and groundwaters by bacteria and other organisms.
- Odours associated with treated effluent may be noticeable to golfers.
- Nitrate contamination of groundwater supplies.
- Unsightly algal and weed growths in reservoirs and ponds.

These concerns should be addressed by secondary treatment and disinfection of effluent that will be subjected to prolonged storage before it is reused for golf course irrigation.

Prolonged storage is expected to remove the slight musty smell of fresh secondary effluent, which might be noticeable and distasteful to some golfers.

Nitrogen and phosphorus are chemical nutrients that are applied as a part of turf grass management. These nutrients, usually provided by commercial fertilizers, may be replaced in part by use of treated effluent.

If nutrient application in commercial fertilizers or treated effluent is properly managed, there is little potential for unsightly and possibly odourous algal and weed growths in lakes, and ponds or of nitrate contamination of groundwater.

If nitrogen is applied at a rate that exceeds the ability of the plant and soil system to contain it or convert it to nitrogen gas, the excess nitrogen may pass through the surface soil and into groundwater. High concentrations of nitrogen, particularly in the form of nitrate, are considered a health hazard in drinking water.

If treated effluent is impounded in an open reservoir a water quality maintenance program which should include one or more of the following measures, is needed:

- Screening or filtration to remove solids, such as algal growths, to reduce maintenance of sprinkler systems.
- Control or prevention of algal growth by an algicide, or a light inhibitor such as blue dye.
- A mixing system.
- Rechlorination to maintain a residual in the distribution system.

Adequate circulation and aeration are necessary for algal and odour control. Aeration can be provided by fountains, air injection, waterfalls, or constructed wetlands.

Algae and weeds in ponds are concerns that can be addressed by assuring that nutrients in fertilizers or effluent are applied to or washed into these bodies.

11.4.2 Soils & Vegetation

The quality of water that is applied in irrigation is an obvious concern to those responsible for management of the soils and vegetation on which a golf course depends.

Irrigation water quality parameters that are of concern include:

- pH.
- Carbonate.
- Bicarbonate.
- Calcium.
- Magnesium.
- Sodium.
- Potassium.
- Conductivity.
- Boron.
- Chloride.
- Sulphate.
- Adjusted SAR.

Bicarbonates and carbonates both increase pH, and are a source of alkalinity, which may affect the water and soil. If the total concentration of bicarbonates plus carbonates exceeds 150 mg/L the resulting increase in pH may affect nutrient availability. Options used to offset this effect include use of acid-forming nitrogen fertilizers, sulphur addition to the soil, or acid injection into the irrigation water.

The adjusted SAR is based on the ratio of sodium to calcium + magnesium in the water. Excess sodium replaces calcium on soil exchange sites, which can result in soil compaction and reduce infiltration into the soil. The usual recommendation for a SAR above 10 is application of calcium, usually a gypsum, and excess irrigation to leach the sodium.

Conductivity is a measure of total soluble salts, or salinity of the water. While most turf species used on golf courses are reasonably salt tolerant, ground covers, ornamental plants, trees, and shrubs may be affected if salt concentrations are too high. There is no recommended restriction on use of irrigation water if the conductivity is less than 3.0 mmhos/cm.

Boron, chloride, and sulphate may be toxic to plants if concentrations are too high. Concentration of boron higher than 1.0 to 2.0 mg/L (0.33 mg/L for some ornamental plants), and of chloride plus sulphate above 250 to 400 mg/L, are considered excessive.

Other parameters that may require regular or occasional measurement are nitrogen, phosphorus, SS, and heavy metals. An excess of inorganic or organic nitrogen may require careful control of fertilizer application. Phosphorus, and nitrogen, may alter the hydraulic properties of soils, or clog sprinkler head openings. If concentrations of heavy metals build up in soils they may complex with phosphorus and other elements and make them unavailable to plants.

11.4.3 Planning

The following is a checklist for use of treated effluent for golf course irrigation:

- Sampling soils:
  - Sample soils well in advance of conversion to effluent (to track effects of the change).
  - Sample from difference parts of the course (tees, greens, fairways, and rough).
  - Sample irrigated soil quarterly to allow for adjustment of watering schedule and use of mitigative measures.
- Water quality:
  - Initial and periodic water analysis.
  - Verify effluent source (noting that if industrial waste is included more undesirable elements may be present).
  - Verify treatment type (more is better).
  - Establish maximum BOD, TSS, and TDS levels in advance.
- Pumping and storage considerations include:
  - Existing pumped or gravity supply need for additional pumping.
  - Form and amount of storage.
  - Need for algae control.
  - Possible use of fresh water for greens, trees, ornamental lakes, and sensitive plants.
- Miscellaneous:
  - Operation, maintenance, and safety issues to be considered.
- Signage:
  - Notification on course and score cards that treated effluent is being used.
  - Time of day of irrigation.

11.4.4 Design

Design considerations related to irrigation with treated effluent include:

- Screening and/or filtration of stored effluent to avoid clogging of the irrigation system.
- If acid injection is involved, consideration of corrosion effects.
- Avoidance of cross-connections between potable and non-potable water systems.
- Labelling and colour-coding of non-potable pipes and equipment.
- Provision of flush valves at low spots and dead ends to allow removal of debris.
- Location of sprinkler heads to avoid contamination of drinking fountains, canteens, food and drink machines, etc.

Sprinkler head location may also have to consider contamination of, or nutrient addition to, water hazards.

Reasons for treated effluent storage include:

- To balance supply and demand.
- To supplement treated effluent with other source.
- To contain excess non-potable water.
- A combination of the above.

Seasonal storage may be required where no alternative effluent disposal method is available.

11.4.5 Management Concerns

Issues that may concern a golf course superintendent include:

- Many older greens have low infiltration rates, and require close attention to avoid turf failure, the risk of which may be increased if there is a possibility that use of treated effluent may further reduce infiltration capacity and require reconstruction of greens.
- If the level of the nutrients in the treated effluent is high, and especially if it is variable, staff will lose the ability to carefully control rates of nutrient application.
- If the application of treated effluent only at night results in a shortened application period, application rates will be increased, and the capacity of pumps and piping may be inadequate.
- If the course is committed to accept and use a certain amount of treated effluent and there is no alternative use for or disposal of water in excess of that used for irrigation, the superintendent may be forced to overwater the course (i.e., irrigation will be based on effluent disposal needs instead of proper golf course management).

Management functions include:

- Public relations: member and player concerns that must be satisfied include odours, course appearance, legal liability, health risks, and adjacent property values.
- Design and construction administration: plan checking, inspection, and record drawings.
- Operation and maintenance: monitoring and testing, staff, player, and public safety.

11.5 References

Canadian Council of Minister of the Environment, *Canadian Environmental Quality Guidelines*, 2005. <https://ccme.ca/en/summary-table>.

Centre for Water Resources Studies, Dalhousie University. *Reuse of Renovated Municipal Wastewater for Golf Course Irrigation*. Prince Edward Island Department of Technology and Environment, 1999. <http://www.gov.pe.ca/photos/original/ee_wasteirr_rep.pdf>.

Environment Canada. *Manual for Land Application of Treated Municipal Wastewater and Sludge*, EPS 6-EP-84-1. Environment Canada, 1984.

Nova Scotia Environment and Labour. *Guidelines for Land Application and Storage of Biosolids in Nova Scotia*. Nova Scotia Environment, 2004.

Ontario Ministry of the Environment,. *Design Guidelines for Sewage Works*. Government of Ontario, 2019. <https://www.ontario.ca/document/design-guidelines-sewage-works-0>.

Tchobanoglous, George, Burton, Franklin L., Stensel, H. David, and Metcalf & Eddy. *Wastewater Engineering: Treatment and Reuse*. 4th Ed. / Revised by George Tchobanoglous, Franklin L. Burton, H. David Stensel. ed. McGraw-Hill Series in Civil and Environmental Engineering. Boston; Montreal: McGraw-Hill, 2003.

Tchobanoglous, George, Burton, Franklin L., Stensel, H. David, and Metcalf & Eddy. *Wastewater Engineering: Treatment and Reuse*. 4th Ed. / Revised by George Tchobanoglous, Franklin L. Burton, H. David Stensel. ed. McGraw-Hill Series in Civil and Environmental Engineering. Boston; Montreal: McGraw-Hill, 2003.

Wastewater Committee of the Great Lakes - Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers. 2014. *Recommended Standards for Wastewater Facilities*. Health Research, Inc., Health Education Services Division. <https://www.health.state.mn.us/communities/environment/water/docs/tenstates/tenstatestan2014.pdf>.

Table of Contents

Chapter 12 Biosolids Management

12.1 General

Bio-solids handling, treatment and disposal must be considered as an integral part of the overall management of sanitary wastewater. The following is a summary of handling, treatment options and the various process and treatment requirements best suited to the option selected. Re-use and recovery alternatives of bio-solids are also included as disposal options. System Owners need to consult with their provincial Regulatory Authorities in determining the appropriate disposal method for their application.

Bio-solids are primarily organic materials produced during the treatment of domestic wastewater sludge, which have been further treated (stabilized) to reduce pathogen content. Due to their nutrient content, bio-solids can be applied to land as a fertilizer or soil amendment, a process which is referred to as land application. Land application of bio-solids can be beneficial by improving crop production and soil properties, reducing requirements for inputs such as fertilizers and irrigation, reclaiming lands (strip mines, quarries, gravel pits, etc.), and enriching forest lands.

Stabilization reduces pathogen concentration, helps minimize odour generation, and reduces vector attraction potential.

12.1.1 Definitions

Aerobic Digestion: the degradation of organic matter brought about through the action of micro-organisms in the presence of oxygen for purposes of stabilization, volume reduction, and pathogen reduction.

Agricultural Land: land on which food, feed, or fiber crops are grown. This includes range land and/or land used as pasture.

Agronomic Rate: the application rate designed to provide the amount of nutrients needed by a crop or vegetation. The goal is to match the needs so the amount of nutrient leaching into the water table can be minimized.

Alkaline Stabilization: see “lime stabilization”.

Anaerobic Digestion: the degradation of organic matter brought about through the action of micro-organisms in the absence of oxygen for purposes of stabilization and pathogen reduction (mesophilic operating range 35 to 38°C. Thermophilic operating range greater than 55°C).

Application Site: see “land application site”.

Beneficial Use: taking advantage of the nutrient content and soil conditioning properties of a bio-solids product to supply some or all of the fertilizer needs of an agronomic crop or for vegetative cover (in land reclamation, silviculture, landfill cover, or similar ventures).

Bio-solids: an organic, stabilized material produced during the treatment of domestic wastewater (some plants may also receive commercial and industrial components) in a WWTP or stabilization lagoon and rendered suitable for beneficial use. They include the solid, semi-solid, and liquid residue removed from primary, secondary, or advanced wastewater treatment processes, but do not include screenings and grit normally removed during the preliminary treatment stages of these processes. Bio-solids differ from wastewater sludges in that they have been treated to reduce pathogen content.

Composting: a stabilization process where organic material undergoes biological degradation to a stable end product. Approximately 20% to 30% of the volatile solids are converted to carbon dioxide and water. Enteric pathogenic organisms are destroyed during this process.

Dewater: increase of the solids concentration of bio-solids and sludges to a cake like consistency generally greater than 15% solids.

Heat Drying: heat drying of bio-solids involves the supply of auxiliary heat to mechanical drying processes in order to increase the vapour holding capacity of the ambient air and to provide the latent heat necessary for evaporation (> 80°C).

Heat Treatment: heat treatment is a continuous process in which bio-solids are heated in a pressure vessel to temperatures up to 260°C for approximately 30 minutes. This serves as both a stabilization process and a conditioning process.

Land Application: the spreading of bio-solids to any one field following the agronomic rate specified in the Nutrient Management Plan (NMP) that has been prepared by a qualified Nutrient Management Planner.

Land Application Site: an area of land (covered by an approval) on which bio-solids are applied to condition the soil, fertilize crops, or promote vegetative growth.

Lime Stabilization: a process in which sufficient lime or other alkaline material is added to bio-solids to produce a highly alkaline sludge (pH of 12 after 2 hours of contact). Also called alkaline stabilization.

Nutrient: any substance that is required for plant growth. This generally refers to nitrogen, phosphorus, potassium, and other essential and trace elements.

Nutrient Management Planner: a professional agrologist that has completed an appropriate course of study that includes nutrient management planning.

Pasteurization: the process in which bio-solids are heated to 70°C for 30 minutes to destroy pathogens.

Pathogens: organisms such as bacteria, protozoa, viruses, and parasites causing disease in humans and animals.

Sludge: the solid, semi-solid, or liquid residue generated during the wastewater treatment process.

Soil Amendment: anything that is added to the soil to improve its physical or chemical condition or plant growth.

Stabilize: to make the organic or volatile portion of the sludge less putrescible, less odorous, and to decrease the vector attraction potential and concentration of pathogenic microorganisms.

Stabilization Lagoon: a facultative (both aerobic and anaerobic), aerobic or anaerobic lagoon capable of degrading the organic matter in wastewater through the action of microorganisms in the presence of oxygen (aerobic) or absence of oxygen (anaerobic) for the purposes of stabilization, volume reduction and pathogen reduction.

Vector Attraction: the characteristic of bio-solids that attracts rodents, flies, mosquitoes, or other pests and organisms capable of transporting infectious agents, such as pathogens.

12.2 Sludge Treatment Process Selection

The selection of sludge handling unit processes should be based upon at least the following considerations:

- Regulatory requirements.
- Local land use.
- System energy requirements.
- Cost effectiveness of sludge thickening and dewatering.
- Equipment complexity and staffing requirements.
- Adverse effects of heavy metals and other sludge components upon the unit processes.
- Sludge digestion or stabilization requirements, including appropriate pathogen and vector attraction reduction.
- Side stream or return flow treatment requirements (e.g., digester or sludge storage facilities supernatant, dewatering unit filtrate, and wet oxidation return flows).
- Sludge storage requirements.
- Methods of ultimate disposal.
- Back-up techniques of sludge handling and disposal.

12.3 Sludge Conditioning

Sludge thickening and dewatering operations (depending on the process used), are highly dependent on sludge conditioning for their effective operation. Sludge conditioning affects the solids concentration of the thickened or dewatered sludge, as well as the capture efficiency. There are two approaches that can be used. Sludge can be conditioned by chemical methods, including the addition of coagulants and/or polymers, or by physical methods such as heat treatment or the addition of fly ash. The method will not only differ in its effect on the thickening or dewatering process but will have different effects on subsequent sludge handling operations, and on the treatment plant itself due to recycle streams.

12.3.1 Chemical Conditioning

12.3.1.1 Chemical Requirements

Chemical conditioning methods involve the use of organic or inorganic flocculants to promote the formation of a porous, free draining cake structure. The ranges of some chemical conditioning requirements are outlined in Table 12.1.

Table 12.1: Some Chemical Conditioning Requirements

Sludge	$FeCl_3$ (kg/tonne Dry Solids)	$Ca(OH)_2$ (kg/tonne Dry Solids)	Polymers (kg/tonne Dry Solids)
RP	10 - 30	0 - 150	1.5 - 2.5
R (P + TF)	30 - 60	0 - 150	2.0 - 5.0
R (P + AS)	40 - 80	0 - 150	3.0 - 7.5
AS	60 - 100	50 - 1,500	4.0 - 12.5
DP	20 - 30	30 - 80	1.5 - 4.0
D (P + TF)	40 - 80	50 - 150	3.0 - 7.5
D (P + AS)	60 - 100	50 - 150	3.0 - 10.0

Key:
R = Raw
P = Primary
TF = Trickling Filter
AS = Activated Sludge
D = Digested

12.3.1.2 Laboratory Testing

The selection of the most suitable chemical(s) and the actual dosage requirements for sludge conditioning should be determined by a combination of pilot and full-scale testing and optimization.

Laboratory testing should only be used to narrow down the selection process and to arrive at approximate dosage requirements. Generally, laboratory testing will yield dosage requirements within 15% of full-scale needs.

12.3.1.3 Conditioning Chemicals

12.3.1.3.1 General

With most thickening operations and with belt filter press dewatering operations the most commonly used chemicals are polymers. For dewatering by vacuum filtration, ferric salts, often in conjunction with lime, are most commonly used. For centrifuge dewatering, chemical conditioning using polymers is most prevalent, with metal salts being avoided mainly due to corrosion problems. The ultimate disposal methods may also have an effect on the choice of conditioning chemicals. For instance, lime and ferric compounds should be avoided with incineration options.

12.3.1.3.2 Iron or Aluminum Salts

Most raw sludges can be filtered with ferric salts alone, although digested sludge will require the addition of lime with the ferric salt. The lime to ferric chloride ratio is typically 3-to-1 to 4-to-1 for best results. If metallic salts are used without lime, the resulting low pH sludge will be highly corrosive to carbon steel and should require materials such as plastic, stainless steel, or rubber for proper handling.

12.3.1.3.3 Lime

Hydrated limes, both the high calcium and dolomitic types, can be used for sludge conditioning in conjunction with metal salts or alone.

12.3.1.3.4 Polymers

Polymers used for sludge conditioning are long-chain water-soluble organic molecules of high molecular weight. They are used in wastewater suspensions to cause flocculation through adsorption. Equipment for polymer addition must be able to withstand potential corrosion.

12.3.1.3.5 Chemical Feed System

The chemical feed system should be paced at the rate of sludge flow to the dewatering unit. The chemical feed system should be either close to the dewatering unit or controllable from a point near the dewatering unit. Sufficient mixing should be provided to disperse the conditioner throughout the sludge. The chemical feed rates should allow for at least a 10-to-1 range of chemical flow to the dewatering unit.

12.3.2 Heating Conditioning

12.3.2.1 General

Heat conditioning of sludge consists of subjecting the sludge to high levels of heat and pressure. Heat conditioning can be accomplished by either a non-oxidative or oxidative system. The high temperatures cause hydrolysis of the encapsulated water-solids matrix and lysing of the biological cells. The hydrolysis of the water matrix destroys the gelatinous components of the organic solids and thereby improves the solid-liquid separation characteristics.

12.3.2.2 Operating Temperatures & Pressures

Typical operating temperatures range from 175 to 205°C. Operating pressures range from 1,700 to 2,800 kPa (247 to 408 psi). Typical sludge detention times vary between 15 and 40 minutes.

12.3.2.3 Increase in Aeration Tank Organic Loading

Although the heat conditioning system has been proven to be an effective sludge conditioning technique for subsequent dewatering operations, the process results in a significant organic loading to the biological treatment process of the WWTP if supernatant is returned to the bioreactor. This is due to the solubilization of organic matter during the sludge hydrolysis. This liquor can represent 25 to 50% of the total loading on the secondary treatment process and allowances must be made in the treatment plant design to accommodate this loading increase.

12.3.2.4 Design Considerations

12.3.2.4.1 Materials

Heat conditioning results in the production of extremely corrosive liquids requiring the use of corrosion-resistant materials for the liquid handling.

12.3.2.4.2 Sludge Grinding

Sludge grinders should be provided to macerate the sludge to a particle size less than 6.0 mm to prevent fouling of the heat exchangers.

12.3.2.4.3 Feed Pumps

Feed pumps should be capable of discharging sludge at pressures of 1,400 to 2,800 kPa (203 to 406 psi) and must be resistant to abrasion.

12.3.2.4.4 Heat Exchangers

The efficiency of the heat exchangers is dependent on the transfer coefficients and the temperature differences of the incoming and outgoing sludges.

12.3.2.4.5 Reaction Vessel

The reaction vessel should be of sufficient volume to provide for a sludge detention time of 15 to 40 minutes. The detention time depends on the sludge characteristics, temperature and the level of hydrolysis required.

12.3.2.4.6 Hot Water Recirculation Pump

The hot water recirculation pump should be capable of handling hot water at the maximum design temperature.

12.3.2.4.7 Odour Control

Heat conditioning, particularly the non-oxidative process, can result in the production of odorous gases in the decant tank. If ultimate sludge disposal is via incineration, these gases can be incinerated in the upper portion of the furnace. If incineration is not a part of the sludge handling process, a catalytic or other type of oxidizing unit should be used.

12.3.2.4.8 Solvent Cleaning

Scale formation in the heat exchangers, pipes, and reaction vessel require acid washing equipment to be provided.

12.3.2.4.9 Piping

All the high-pressure piping for the sludge heat conditioning system should be tested at a pressure of 3,500 kPa (508 psi). Low pressure piping should be tested at 1.5 times the working pressure or 1,400 kPa (203 psi), whichever is greater.

12.3.2.4.10 Decant Tank

The decant tank functions as a storage and sludge consolidation unit. The tank should be covered and provided with venting and a deodorization arrangement. The tank should be designed using loadings of 245 $kg/m^2 \cdot d$ for primary sludge and 145 $kg/m^2 \cdot d$ for biological sludges. The underflow will range from 1.0 to 1.5% TS.

12.3.2.5 Laboratory Testing

Since process efficiency is dependent on achieving a degree of solubilization (hydrolysis) that reduces the specific resistant to an acceptable range, batch testing with a laboratory autoclave should be employed. This procedure permits accurate control of the time and temperature functions affecting the level of hydrolysis. The level of solubilization is determined from the loss of TSS during heat treatment.

12.3.3 Addition of Admixtures

Another common form of physical conditioning is the addition of admixtures such as fly ash, incinerator ash, diatomaceous earth, or waste paper. These conditioning techniques are most commonly used with filter presses. The admixtures when added in sufficient quantities produce a porous lattice structure in the sludge which results in decreased compressibility and improved filtering characteristics. When considering such conditioning techniques, the beneficial and detrimental effects of the admixture on such parameters as overall sludge mass, calorific value, etc., must be evaluated along with the effects on improved solids content.

12.4 Sludge Thickening

12.4.1 General

12.4.1.1 Applicability

As the first step of sludge handling, the need for sludge thickeners to reduce the volume of sludge should be considered.

The design of thickeners (gravity, DAF, centrifuge, and others) should consider the type and concentration of sludge, the sludge stabilization processes, storage requirements, the method of ultimate sludge disposal, chemical needs, and the cost of operation. Particular attention should be given to the pumping and piping of the concentrated sludge and possible onset of anaerobic conditions. Sludge thickening to at least 5% solids prior to transmission to digesters should be considered.

Wherever possible, pilot-plant and/or bench-scale data should be used for the design of sludge thickening facilities. With new plants, this may not always be possible, and, in such cases, empirical design parameters must be used. The following subsections outline the normal ranges for the design parameters of such equipment.

In considering the need for sludge thickening facilities, the designer should evaluate the economics of the overall treatment processes, with and without facilities for sludge water content reduction. This evaluation should consider both capital and operating costs of the various plant components and sludge disposal operations affected.

12.4.1.2 Multiple Units

With sludge thickening equipment, multiple units will generally be required unless satisfactory sludge storage facilities or alternate sludge disposal methods are available for use during periods of equipment repair. Often the need for full standby units will be unnecessary if the remaining duty units can be operated for additional shifts in the event of equipment breakdown.

12.4.1.3 Thickener Location

Sludge thickening can be employed in the following locations in a WWTP:

- Prior to digestion for raw primary, secondary sludge, or mixed sludges.
- Prior to dewatering facilities.
- Following digestion for sludges or supernatant.
- Or following dewatering facilities for concentration of filtrate, decant, centrate, etc.

Where thickeners are to be housed, ventilation and electrical requirements must be consistent with the area classification as determined by the latest edition of *NFPA 820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities*.

12.4.2 Thickening Methods & Performance With Various Sludge Types

The commonly employed methods of sludge thickening and their suitability for the various types of sludge are shown in Table 12.2. In selecting a design figure for the thickened sludge concentration, the designer should keep in mind that all thickening devices are adversely affected by high SVIs and benefited by low SVIs in the WAS. Thickening targets should also consider digestion needs. For example, pre-thickening prior to aerobic digestion can lead to odour and foaming issues. The ranges of thickened sludge concentrations provided in

Table 12.2 assumes an SVI of approximately 100.

Table 12.2: Sludge Thickening Methods & Performance with Various Sludge Types

Thickening Method	Sludge Type	Performance Expected
Gravity	Raw primary	• Good (8 - 10% solids).
	Raw primary and waste activated	• Poor (5 - 8% solids).
	Waste activated	• Very poor (2 - 3% solids (better results reported for oxygen excess activated sludge)).
	Digested primary	• Very good (8 - 14% solids).
	Digested primary and waste activated	• Poor (6 - 9% solids).
DAF	Waste activated (not generally used for other sludge types)	• Good (4 - 6% solids and ≥ 95% solids capture with flotation aids).
Centrifugation	Waste activated	• 8 - 10% and 80 - 90% solids capture with basket centrifuges. • 4 - 6% and 80 - 90% solids capture with disc-nozzle centrifuges. • 5 - 8% and 70 - 90% solids capture with solid bowl centrifuges.

12.4.3 Sludge Pre-Treatment

Wherever thickening devices are being installed, special consideration must be given to the need for sludge pre-treatment in the form of sludge grinding to avoid plugging pumps, lines, and thickening equipment. Sludge conditioning by chemical conditioning is also considered as a type of pre-treatment.

12.4.4 Gravity Thickening

12.4.4.1 Process Application

Gravity thickening is primarily used for primary sludge, and mixtures of primary and WAS. Due to the better performance of other methods for WAS, gravity thickening has limited application for such sludges.

12.4.4.2 Design Criteria

12.4.4.2.1 Tank Shape

The gravity thickener should be circular in shape.

12.4.4.2.2 Tank Dimensions

Typical tank diameters should range between 21 and 24 m. Sidewater depth should be between 3 and 3.7 m.

12.4.4.2.3 Floor Slope

The acceptable range for gravity sludge thickener floor slopes is 2-to-12 to 3-to-12.

12.4.4.2.4 Solids Loadings

The type of sludge should govern the design value for solids loading to the gravity thickener.
Table 12.3 outlines recommended solids loading values.

Table 12.3: Solids Loading on Gravity Thickeners for Various Sludge Types

Type of Sludge	Solids Load (kg/m^2-day) Acceptable Range
Primary	95 - 120
Waste activated	12 - 40
Modified activated	50 - 100
Trickling filter	40 - 50

Solids loading for any combination of primary sludge and WAS should be based on a weighted average of the above loading rates.

Use of metal salts for phosphorus removal may affect the solids loading rates.

12.4.4.2.5 Dilution

Improved thickening is achieved by diluting sludge to 0.5 to 1% solids because that dilution reduces the interface between the settling particles. Primary wastewater effluent or secondary effluent may be utilized to dilute sludge before thickening.

12.4.4.2.6 Hydraulic Overflow Rate

The hydraulic overflow rate should be kept sufficiently high to prevent septic conditions from developing in the thickener. The acceptable ranges for overflow rates are as follows:

Primary sludge	0.28 - 0.38 $L/m^2 \cdot s$
Secondary sludge	0.22 - 0.34 $L/m^2 \cdot s$
Mixture	0.25 - 0.36 $L/m^2 \cdot s$

12.4.4.2.7 Sludge Volume Ratio

The Sludge Volume Ratio (SVR) is defined as the volume of the sludge blanket divided by the daily volume of sludge (underflow) pumped from the thickener. Though deeper sludge blankets and longer SVR are desirable for maximum concentrations, septic conditions due to anaerobic biodegradation on warmer months limit the upper values of SVR to about 1 day.

Recommended SVR values range is 0.3 to 1 day during warmer months and 0.5 to 2 days during colder months.

12.4.4.2.8 Hydraulic Retention Time

A minimum of 6 hours of HRT is required. For maximum compaction of the sludge blanket, 24 hours is recommended.

During peak conditions, the retention time may have to be shortened to keep the sludge blanket level below the overflow weirs, thus, preventing excessive solids carry-over.

12.4.4.2.9 Sludge Underflow Piping

The length of suction lines should be kept as short as possible. Consideration should be given to the use of dual sludge withdrawal lines.

12.4.4.2.10 Chemical Conditioning
Provision should be made for the addition of conditioning chemicals into the sludge influent lines (polymers, ferric chloride or lime are the most likely chemicals to be used to improve solids capture).

12.4.4.2.11 Mechanical Rake
The mechanical rake should have a tip speed of 50 to 100 mm/second. The rake should be equipped with hinged-lift mechanisms when handling heavy sludges such as lime treated primary sludge. The use of a surface skimmer is recommended.

12.4.4.2.12 Overflow Handling
The normal quality of thickener overflow (also known as thickener overhead or supernatant) is about the same as raw wastewater quality. Consequently, returning the overflow to primary settling tank or aeration tank should not present any operational problem.

Direct recycling of thickener overflow to the grit chamber, primary settling tank, trickling filter, RBC, or aeration tank is permitted. The supernatant should not be discharged into the secondary settling tank, disinfection tank, sewer outfall, or receiving water.

12.4.5 Dissolved Air Flotation

12.4.5.1 Applicability
Unlike heavy sludges, such as primary and mixtures of primary and WASs, which are generally most effectively thickened in gravity thickeners, light WASs can be successfully thickened by DAF. In general, air flotation thickening can be employed whenever particles tend to float rather than sink. These procedures are also applied if the materials have a long subsidence period and resist compaction for thickening by gravity.

The advantages of air flotation compared with gravity thickeners for WASs include its reliability, production of higher sludge concentrations, and better solids capture. Its disadvantages include the need for greater operating skill and higher operating costs.

12.4.5.2 Pilot Scale Testing
Experience has shown that flotation operations cannot be designed on the basis of purely mathematical formulations or by the use of generalized design parameters and, therefore, some bench-scale and/or pilot-scale testing will be necessary.

12.4.5.3 Design Parameters
The following design parameters are given only as a guide to indicate the normal range of values experienced in full-scale operations.

12.4.5.3.1 Recycle Ratio
The recycle ratio varies with suppliers and typically falls between 0 and 500% of the influent flow. Recycled flows may be pressurized up to 520 kPa (75 psi).

12.4.5.3.2 Air to Solids Weight Ratio
Typical air to solids weight ratios should be between 0.02 and 0.05.

12.4.5.3.3 Feed Concentration

Feed concentration of activated sludge (including recycle) to the flotation compartment should not exceed 5,000 mg/L.

12.4.5.3.4 Hydraulic Feed Rate

Where the hydraulic feed rate includes influent plus recycle, the flotation units should be designed hydraulically to operate in the range of 0.3 to 1.5 L/m^2·s. A maximum HLR of 0.5 L/m^2·s should be adhered to when no coagulant aids are used to improve flotation. The feed rate should be continuous rather than on-off.

12.4.5.3.5 Solids Loading

Without any addition of flocculating chemicals, the solids loading rate for activated sludge to a flotation unit should be between 40 and 100 kg/m^2·d. With the proper addition of flocculating chemicals, the solids loading rate may be increased to 240 kg/m^2·d. These loading rates will generally produce a thickened sludge of 3 to 5% TS.

12.4.5.3.6 Chemical Conditioning

Chemicals should be fed directly to the mixing zone of the feed sludge and recycle flow. Most installations use chemical conditioning with polymers to achieve provide more economical operation. Polymer feed rate in the range of 0 to 25 g/kg of dry solids.

12.4.5.3.7 Detention Time

Detention time is not critical provided particle rise rate is sufficient and horizontal velocity in the unit does not produce scouring of the sludge blanket.

12.4.5.4 Thickened Sludge Withdrawal

The surface skimmer should move thickened sludge over the dewatering beach into the sludge hopper. Either positive displacement, or centrifugal pumps which will not air bind should be used to transfer sludge from the hopper to the next phase of the process. When selecting pumps, the maximum possible sludge concentrations should be taken into consideration.

12.4.5.5 Bottom Sludge

A bottom collector to move settled sludge into a hopper must be provided. Sludge removal from the hopper may be by gravity or pumps.

12.4.6 Centrifugation

12.4.6.1 Types of Centrifuges

Three types of centrifuges may be utilized for sludge thickening. These include the solid bowl conveyor, disc-nozzle, and basket centrifuges.

12.4.6.2 Applicability

To date, there has been limited application of centrifuges for sludge thickening, despite their common use for sludge dewatering. As thickening devices, their use has been generally restricted to WASs.

General design considerations are as follows:

- Centrifugal thickening operations can have substantial maintenance and operating costs.
- Where space limitations, or sludge characteristics make other methods unsuitable, or where high-capacity mobile units are needed, centrifuges have been used.
- Thickening capacity, thickened sludge concentration and solids capture of a centrifuge are greatly dependent on the SVI of the sludge.
- Early experience with disc nozzle-type centrifuges found that clogging of sludge discharge nozzles resulted in frequent maintenance, recently pre-treatment has helped to alleviate these concerns.
- Basket type centrifuges have seen limited use due to their low capacity and batch operation, and as such their use has been restricted to small plants.

12.4.6.3 Solids Recovery

The most suitable operating range is generally 85 to 95% solids recovery.

12.4.6.4 Polymer Feed Range

A polymer feed range of 0 to 4.0 g/kg of dry solids is generally acceptable.

12.4.7 Gravity Belt Thickener

12.4.7.1 Applicability

Gravity Belt Thickeners (GBTs) have been used on both primary, waste activated, and mixtures of primary and WASs. Their use stems from the application of belt filter presses for sludge dewatering.

General design considerations as follows:

- Performance of the GBTs is subject to upstream conditions at the treatment plant, the better the settling of solids at the facility, the better the GBT will function and potentially at lower chemical dosages.
- Adequate attention should be given to transporting the thickened solids, in particular for handling the maximum solids content expected.
- Prior to digestion, adequate mixing or blending of thickened solids with other solids is required.
- Plows on the gravity belt turn and distribute the thickened solids to allow for water to drain through the belt fabric. The number and location should be adjustable for each type of sludge being thickened.
- Chemical addition and mixing equipment are important, as are multiple injection points.
- Gravity belt thickeners should have an air handling system to maintain a safe working environment; this could include a complete enclosure with exhaust, odour control, inspection door, and access for cleaning.
- Gravity belt thickeners should have a curb around them, and floors sloped to drains so that operators can properly clean the equipment.
- Metering of solids into and out of the equipment is important.
- Thickened solids need to be designed to move all expected material and avoid accumulation and overload.
- Due to height of equipment, an elevated walkway will probably be needed to operate and maintain the equipment.
- Scum (grease) should not be placed on the GBT because blinding of the fabric can create problems.

12.4.8 Rotary Drum Thickener

12.4.8.1 Applicability

Rotary Drum Thickeners (RDTs) have been used on both primary, waste activated, and mixtures of primary and WAS's.

Design considerations are similar to GBTs.

12.5 Sludge Dewatering

12.5.1 General

Sludge dewatering will often be required at WWTPs prior to ultimate disposal of sludges or as a prelude to further treatment or stabilization. Since the processes differ significantly in their ability to reduce the water content of sludges, the ultimate sludge disposal method will generally have a major influence on the dewatering method most suitable for a particular WWTP. Also of influence will be the characteristics of the sludge requiring dewatering, that is, whether the sludge is raw or digested, whether the sludge contains WAS, or whether the sludge has been previously thickened. With raw sludge, the freshness of the sludge will have a significant effect on dewatering performance (septic sludge will be more difficult to dewater than fresh raw sludge).

Similar to thickening systems, dewatering facilities may require sludge pre-treatment in the form of sludge grinding to avoid plugging pumps, lines and plugging or damaging dewatering equipment. Ventilation and electrical requirements must be consistent with the area classification as determined by the latest edition of *NFPA 820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities*.

In evaluating dewatering system alternatives, the designer must consider the capital and operating costs, including labour, parts, chemicals, and energy, for each alternative as well as for the effects which each alternative will have on the wastewater treatment and subsequent sludge handling and ultimate sludge disposal operations.

In considering the need for sludge dewatering facilities, the designer should evaluate the economics of the overall treatment processes, with and without facilities for sludge water content reduction. This evaluation should consider both capital and operating costs of the various plant components and sludge disposal operations affected.

Wherever possible, pilot-plant and/or bench-scale data should be used for the design of dewatering facilities. With new plants, this may not always be possible and, in such cases, empirical design parameters must be used. The following subsections outline the normal ranges for the design parameters of such equipment.

For calculating dewatering design sludge handling needs, a rational basis of design for sludge production from sludge stabilization processes should be developed and provided to the Regulatory Authorities for approval on a case-by case basis. In lieu of actual sludge production data, an overall mass balance should be prepared to account for anticipated sludge production from each unit process, including the recycle streams.

12.5.2 Sludge Storage

Sludge storage facilities should be provided at all mechanical treatment plants. Appropriate storage facilities may consist of any combination of drying beds, lagoons, separate tanks, additional volume in sludge stabilization units, pad area or other means to store either liquid or dried sludge.

The design should provide for odour control in sludge storage tanks and lagoons including aeration, covering or other appropriate means.

Calculations to establish the number of days of storage should be carried out based on the overall sludge handling and disposal methodology.

12.5.3 Dewatering Process Compatibility with Subsequent Treatment or Disposal Techniques

Table 12.4 outlines the relationship of dewatering to other processes.

Table 12.4: The Relationship of Dewatering to Other Sludge Treatment Processes for Typical Municipal Sludges

Method	Pretreatment Normally Provided		Normal Use of Dewatered Cake			
	Thickening	Conditioning	Landfill	Land Spread	Heat Drying	Incineration
Rotary press	Yes	Yes	Yes	Yes	Yes	Yes
Centrifuge (solid bowl)	Yes	Yes	Yes	Yes	Yes	Yes
Centrifuge (basket)	Variable	Variable	Yes	Yes	No	No
Drying beds	Variable	Not usually	Yes	Yes	No	No
Lagoons	No	No	Yes	Yes	No	No
Filter presses	Yes	Yes	Yes	Variable	Not usually	Yes
Horizontal belt filters	Yes	Yes	Yes	Yes	Yes	Yes

12.5.4 Sludge Drying Beds

12.5.4.1 Pre-Treatment

Sludge should be pre-treated before being air-dried by either one of the following methods:

- Anaerobic digesters.
- Aerobic digesters with provision to thicken.
- Digestion in aeration tanks of extended aeration plants (with long sludge age, greater than about 20 days) preferably with provision to thicken using thickeners, lagoons or by other means.
- Properly designed and maintained oxidation ditches with sludge age longer than about 20 days (preferably after thickening).

12.5.4.2 Chemical Conditioning

The dewatering characteristics can be considerably improved by chemical conditioning of sludge prior to treatment in beds.

Since sludge conditioning can reduce the required drying time to 1/3 or less, of the unconditioned drying time, provision should be made for the addition of conditioning chemicals, usually polymers.

12.5.4.3 Design Criteria

12.5.4.3.1 Factors Influencing Design

The design and operation of sludge drying beds depend on the following factors:

- Climate, including site specific climate change considerations (e.g., potential changes in annual and seasonal precipitation and increasing temperatures).
- Sludge characteristics.
- Pre-treatment (such as conditioning, thickening, etc.).
- Sub-soil permeability.

12.5.4.3.2 Bed Area

Consideration should be given to the following when calculating the bed area:

- The volume of wet sludge produced by existing and proposed processes.
- Dosing depth:
  - For design calculation purposes, a maximum depth of 200 mm should be utilized.
  - For operational purposes, the depth of sludge placed on the drying bed may increase or decrease from the design depth based on the percent solids content, type of digestion utilized, and seasonal variations.
- Total digester volume and other wet sludge storage facilities.
- Degree of sludge thickening provided after digestion.
- The maximum depth of sludge which can be removed from the digester or other sludge storage facilities without causing process or structural problems.
- The time required on the bed to produce a removable cake. Adequate provision should be made for sludge dewatering and/or sludge disposal facilities for those periods of time during which outside drying of sludge on beds in hindered by weather.
- Capacities of auxiliary dewatering facilities.

Sludge drying beds may be designed from basic principles, laboratory tests, and/or pilot plant field studies. Calculations must be presented to the Regulatory Authority supporting any design based on the above methods. In the absence of such calculations the minimum sludge drying bed area should be based on the criteria presented in Table 12.5.

Table 12.5: Sludge Drying Bed Areas

Type of Wastewater Treatment	Area (m^2/capita)		
	Open Beds	Covered Beds	Combination of Open and Covered Beds
Primary plants (no secondary treatment)	0.12	0.10	0.10
Activated sludge (no primary treatment)	0.16	0.13	0.13
Primary and activated sludge	0.20	0.16	0.16

The area of the bed may be reduced by up to 50% if it is to be used solely as a back-up dewatering unit. An increase of bed area by 25% is recommended for paved beds.

12.5.4.3.3 Percolation Type Beds

Pond Bottom
The bottom of the cell should be of impervious material such as clay or asphalt.

Underdrains
Underdrains should be at least 100 mm in diameter laid with open joints. Perforated pipe may also be used. Underdrains should be spaced 2.5 to 3.0 m apart, with a minimum slope of 1%. Underdrains should discharge back to the secondary treatment section of the WWTP. Various pipe materials may be selected provided the material is of suitable strength, corrosion resistant, and appropriately bedded to ensure that pipe is not damaged by sludge removal equipment.

Gravel
The lower course of gravel around the underdrains should be properly graded, should be at least 300 mm in depth, extending at least 150 mm above the top of the underdrains, and should be placed in two or more layers. The top layer, of at least 75 mm in depth, should consist of gravel 3.0 to 6.0 mm in size.

Sand
The top course should consist of 250 to 450 mm of clean coarse sand. The effective size should range from 0.3 to 1.2 mm with a uniformity coefficient of less than 5.0 mm. The finished sand surface should be level.

Additional Dewatering Provisions
Consideration should be given for providing a means of decanting supernatant of sludge placed on the sludge drying beds. More effective decanting of supernatant may be accomplished with polymer treatment of sludge.

12.5.4.3.4 Impervious Type Beds

Paved drying beds should be designed with consideration for space requirements to operate mechanical equipment for removing the dried sludge.

12.5.4.3.5 Location

Depending on prevailing wind directions, a minimum distance of 100 to 150 m should be kept from open sludge drying beds and dwellings, however, the minimum maybe reduced to 60 to 80 m for enclosed beds. The selected location for open beds should be at least 30 m from public roads and 25 m for enclosed beds. The System Owner may be required to spray deodorants and odour masking chemicals whenever there are complaints from the population in the neighbourhood.

12.5.4.3.6 Winter Storage

Alternative methods of disposal should be arranged for the non-drying season which may start as early as October or November and end in April or May. Note that the transitions between seasons is likely to be impacted as monthly precipitation shifts with climate change.

12.5.4.3.7 Dimensions

The bed size generally should be 4.5 to 7.5 m wide with the length selected to satisfy desired bed loading volume.

12.5.4.3.8 Depth of Sludge
The sludge dosing depth should generally be 200 to 300 mm for warm weather operating modes; for winter freeze drying depths of 1.0 to 3.0 m can be used depending upon the number of degree days in winter.

12.5.4.3.9 Number of Beds
Three beds are desirable for increased flexibility of operation. Not less than two beds should be provided.

12.5.4.3.10 Walls
Walls should be watertight and extend 400 to 500 mm above and at least 200 mm below the surface. Outer walls should be extended at least 100 mm above the outside grade elevation to prevent soil from washing on to the beds.

12.5.4.3.11 Sludge Influent
The sludge pipe to the beds should terminate at least 300 mm above the surface and be so arranged that it will drain. Concrete splash plates for percolation type beds should be provided at sludge discharge points. One inlet pipe per cell should be provided.

12.5.4.3.12 Sludge Removal
Each bed should be constructed to be readily and completely accessible to mechanical cleaning equipment. Concrete runways spaced to accommodate mechanical equipment should be provided. Special attention should be given to assure adequate access to the areas adjacent to the sidewalls. Entrance ramps down to the level of the sand bed should be provided. These ramps should be high enough to eliminate the need for an entrance end wall for the sludge bed.

Atlantic Canada climatological conditions may permit three or four cycles (consisting of filling the open bed with digested sludge, drying, and emptying) during the drying season, however, the number of cycles may be increased to approximately ten with covered beds. These values are tentative and subject to revision after field observations and should account for climate change impacts (i.e., potential reductions to drying times from increases in precipitation). Refer to Chapter 2 for further guidance.

12.5.4.3.13 Covered Beds
Consideration should be given to the design and use of covered sludge drying beds.

12.5.5 Sludge Lagoons

12.5.5.1 General
Sludge drying lagoons may be used as a substitute for drying beds for the dewatering of digested sludge. Lagoons are not suitable for dewatering untreated sludges, limed sludges, or sludges with a high strength supernatant because of their odour and nuisance potential. The performance of lagoons, like that of drying beds, is affected by climate such as precipitation and low temperatures which inhibit dewatering. Lagoons are most applicable in areas with high evaporation rates.

Sludge lagoons may also be used as temporary sludge storage facilities, when spreading on agricultural land cannot be carried out due to such factors as wet ground, frozen ground, or snow cover.

Sludge lagoons as a means of dewatering digested sludge will be permitted only upon proof that the character of the digested sludge and the design mode of operation are such that offensive odours will not result. Where sludge lagoons are permitted, adequate provisions should be made for other sludge dewatering facilities or sludge disposal in the event of upset or failure of the sludge digestion process.

12.5.5.2 Design Considerations

The design, operation, and location of sludge lagoons must take into consideration many factors, including the following:

- Possible nuisances:
  - Odours.
  - Appearance.
  - Mosquitos.
- Proximity to:
  - Dwellings.
  - Water supply wells.
  - Watercourses.
  - Property lines.
- Design:
  - Number.
  - Configuration.
  - Retention time.
  - Freeboard.
  - Size.
  - Shape depth.
  - Site grading.
- Loading factors:
  - Solids concentration of digested sludge.
  - Loading rates.
- Operation:
  - Receiving station(s).
  - Monitoring.
  - Sampling.
  - Fencing.
  - Access.
  - Odour control.
  - pH control.
  - Reporting.
  - Contingency planning.
- Soil conditions:
  - Permeability of soil.
  - Need for liner.
  - Stability of berm slopes.
  - Depth to bedrock (see Section 8.6.2.4), etc.
- Groundwater and surface water conditions:
  - Elevation of maximum groundwater level (see Section 8.6.2.4).
  - Direction of groundwater movement.

- Location of monitoring and any drinking water wells and surface water bodies in the area.
- Sludge and supernatant removal:
  - Volumes.
  - Concentrations.
  - Methods of removal.
  - Method of supernatant treatment.
  - Final sludge disposal.
- Climatic processes (including climate change considerations):
  - Evaporation.
  - Rainfall.
  - Freezing.
  - Snowfall.
  - Temperature.
  - Solar radiation.
- Final closure:
  - Rehabilitation.
  - Restoration of the site.

12.5.5.3 Pre-Treatment
Pre-treatment requirements for sludge lagoons are similar to sludge drying beds.

12.5.5.4 Soil & Groundwater Conditions
The soil must be reasonably porous, and the bottom of the lagoons must be at least 1.2 m above the maximum groundwater table. Surrounding areas should be graded to prevent surface water from entering the lagoon. In some critical instances, the Regulatory Authority may require a lagoon to be lined with a synthetic material.

12.5.5.5 Depth
Lagoons should be at least 1.0 m in depth while maintaining a minimum of 0.6 m of freeboard.

12.5.5.6 Seal
Adequate provisions should be made to seal the sludge lagoon bottom and embankments in accordance with the requirements of Section 8.6.6 to prevent leaching into adjacent soils or groundwater. Seal to be protected to prevent damage from sludge removal equipment.

12.5.5.7 Volume
The volume required will depend on local climatic conditions, including climate change considerations (e.g., potential changes in the intensity of precipitation events). Not less than two lagoons should be provided.

12.5.5.8 Location
Consideration should be given to prevent pollution of ground and surface water. Adequate isolation should be provided to alleviate nuisance impact.

12.5.5.9 Cycle Time & Sludge Removal

The cycle time for lagoons varies from several months to several years. Typically, sludge is pumped to the lagoon for 18 months and then the lagoon is rested for 6 months.

Sludge is removed mechanically, usually at a moisture content of about 70%.

12.5.6 Mechanical Dewatering Facilities

12.5.6.1 General

Provisions should be made to maintain sufficient continuity of service so that sludge may be dewatered without accumulation beyond storage capacity. If it is proposed to dewater the sludge by mechanical methods such as rotary press, centrifuges, filter presses or belt filters, a detailed description of the process and design data should accompany the plans.

Unless standby facilities are available, adequate storage facilities should be provided. The storage capacity should be sufficient to handle at least 4 days of sludge production volume.

12.5.6.2 Performance of Mechanical Dewatering Methods

Table 12.6 outlines the solids capture, solids concentrations normally achieved and energy requirements for various mechanical dewatering methods.

Table 12.6: Sludge Dewatering Methods & Performance with Various Sludge Types

Dewatering Method	Solids Capture (%)	Solids Concentrations Normally Achieved (1)	Median Energy Required (MJ/Dry Tonne) (2)
Rotary press	90 - 95	• Raw primary + WAS (25 - 35%). • Digested primary + WAS (15 - 25%). • WAS (13 - 20%).	70 - 80
Filter press	90 - 95	• Raw primary + WAS (30 - 50%). • Digested primary + WAS (35 - 50%). • WAS (25 - 50%).	360
Centrifuge (solid bowl)	95 - 99	• Raw or digested primary + WAS (15 - 25%). • WAS (12 - 15%).	360
Belt filter press	85 - 95	• Raw or digested primary + WAS (14 - 25%). • WAS (10 - 15%).	130

1. Including conditioning chemicals (i.e., polymer), if required.
2. MJ/dry tonne - denotes megajoules per dry tonne of sludge throughput.

12.5.6.3 Number of Units

With sludge dewatering equipment, multiple units will generally be required unless satisfactory sludge storage facilities or alternate sludge disposal methods are available for use during periods of equipment repair. Often the need for full standby units will be unnecessary if the remaining duty units can be operated for additional shifts in the event of equipment breakdown.

12.5.6.4 Ventilation
Adequate facilities should be provided for ventilation of the dewatering area in accordance with area classification as determined by the latest edition of *NFPA 820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities*. The exhaust air should be properly conditioned to avoid odour nuisance.

12.5.6.5 Chemical Handling Enclosures
Lime-mixing facilities should be completely enclosed to prevent the escape of lime dust. Chemical handling equipment should be automated to eliminate the manual lifting requirement.

12.5.6.6 Drainage & Filtrate Disposal
Drainage or filtrate from dewatering units should be returned to the wastewater treatment process at appropriate points and flowrates.

12.5.6.7 Other Dewatering Facilities
If it is proposed to dewater sludge by mechanical means, other than those outlined below, a detailed description of the process and design should accompany the plans.

12.5.6.8 Rotary Presses
Most types of wastewater sludge can be dewatered with rotary presses and the results achieved are generally superior to those of vacuum filters or belt filter presses.

The rotary press is a pressure-controlled device and should be provided with its own inlet pressure controls, outlet pressure controls, feed flow monitoring and polymer feed flow controlling.

The rotary press should be provided with its own flocculation chamber, equipped with a variable-speed impeller.

Flocculated sludge is fed continuously into the channels where it is dewatered.

A channel consists of a pair of rotating screens, coupled to a driving hub, and enclosed by a fabricated steel housing, each housing being completely removable and interchangeable with other channels of the same size and description.

The number of channels needed is determined by the:
- Flow requirements.
- Quality of sludge.
- Cake dryness.
- Filtrate quality.
- Economic considerations.
- Dimensional considerations.
- Maintenance considerations.

A rotary press comprises of at least one or several dewatering channels, mounted on the gear reducer output shaft.

Dewatering channels

The following are the major components to the dewatering channel:

- Filtration elements.
- Filtration wheels.
- Deflector.
- Gland covers and bearings.
- Wash system.

Drive system

The major components to the drive system include:

- Speed reducer.
- Motor.

12.5.6.9 Filter Presses

The capacity of filter presses is greatly affected by the initial solids concentration. With low feed solids, chemical requirements increase significantly. Sludge thickening should, therefore, be considered as a pre-treatment step.

Filter press systems should be designed in accordance with the following guidelines.

Sludge conditioning tank:

Detention time maximum 20 minutes at peak pump rate.

Feed pumps:

Variable capacity to allow pressures to be increased gradually, without underfeeding or overfeeding sludge; pumps should be of a type to minimize floc shear. Pumps must deliver high volume at low head initially and low volume at high head during latter part of cycle. Ram or piston pumps, progressing cavity pumps, or double diaphragm pumps are generally used.

Cake handling:

Filter press must be elevated above cake conveyance system to allow free fall; cake can be discharged directly to trucks, into dumper boxes, or onto conveyors (usually cake breakers may be needed).

Cycle times:

One and a half (1.5) to 6 hours (normally 1.5 to 3 hours).

Operating pressures:

Usually 700 to 1,400 kPa (102 to 203 psi) but may be as high as 1,750 kPa (254 psi). The operating pressure should not exceed 1,000 to 1,050 kPa (145 to 152 psi) if polymer is applied as the conditioning agent.

12.5.6.10 Solid Bowl Centrifuges

The variables of importance for centrifuges include:

- Bowl length/diameter ratio.
- Bowl angle.
- Bowl flow pattern.
- Bowl speed.

- Pool volume.
- Internal conveyor design.
- conveyor speed.

Bowl length/diameter ratios of 2.5 to 4.0 should be provided to ensure adequate settling time and surface area. Bowl angles must be kept shallow.

The bowl flow pattern can be either countercurrent or concurrent. Pool depth can be varied by adjustable weirs.

Increased bowl speed increases the centrifugal forces available for clarification, but the settled solids can become more difficult to remove due to the higher gravitational forces.

Conveyor design and speed will affect the efficiency of solids removal. Differential speed must be kept low enough to minimize turbulence and internal wear yet high enough to provide sufficient solids handling capacity.

For most wastewater sludges, the capacity of the centrifuge will be limited by the clarification capacity (hydraulic capacity) and, therefore, the solids concentration. Increasing the feed solids will increase the solids handling capacity. Thickening should, therefore, be considered as a pre-treatment operation.

Since temperature affects the viscosity of sludges, if the temperatures will vary appreciably (as with aerobic digestion), the required centrifuge capacity should be determined for the lowest temperature expected.

The chemical conditioning agents most commonly used for centrifuges are polymers. Flocculating agents are typically injected at the head of the unit to avoid shearing the floc.

Other general design guidelines for solid bowl centrifuges are as follows:

Feed pump:

- Sludge feed should be continuous.
- Pumps should be variable flow type.
- One pump should be provided per centrifuge for multiple centrifuge systems.
- Chemical dosage should vary with the pump rate.

Sludge pre-treatment:

- Depending upon the wastewater treatment process, grit removal, screening or maceration may be required for the feed sludge stream.

Solids capture:

- Generally 85 to 95% is desirable.

Machine materials:

- Generally, carbon steel or stainless steel.
- Parts subject to wear should be protected with hard facing materials such as a tungsten carbide material.

Machine foundations:

- Foundations must be capable of absorbing the vibratory loads.

Provision for maintenance:

- Sufficient space must be provided around the machine(s) to permit disassembly.
- An overhead hoist should be provided.
- Hot and cold-water supplies will be needed to permit flushing out the machine.
- Drainage facilities will be necessary to handle wash water.

12.5.6.11 Belt Filter Presses

Most types of wastewater sludges can be dewatered with belt filter presses.

Chemical conditioning is generally accomplished with polymer addition.

Solids handling capabilities are likely to range from 50 g/m·s (based on belt width) for excess activated sludge to 330 g/m·s for primary sludge.

12.6 Sludge Pumps & Piping

12.6.1 Sludge Pumps

12.6.1.1 General Sludge Pumping Requirements

Table 12.7 outlines general sludge pumping requirements for various sludge types.

Table 12.7: General Sludge Pumping Requirements

Sludge Source	Slurry (% TS)	Static Head (m)	TDH (m)	Abrasive Service	Duty
Pre-treatment-grit	0.5 - 10.0	0 - 1.5 (Gravity)	1.5 - 3.0	Yes - High	Heavy
Primary sedimentation Unthickened Thickened	 0.2 - 2.0 4.0 - 10.0	 3 - 12 3 - 12	 10 - 200 12 - 25	 Yes Yes	 Medium Heavy
Secondary sedimentation (for recirculation)	0.5 - 2.0	1 - 2	3 - 4.5	No	Light
Secondary sedimentation (for thickening)	0.5 - 2.0	1.2 - 2.4	3 - 4.5	No	Light
Thickener Underflow	5 - 10 5 - 10	6 - 12 60 - 120**	25 - 45 75 - 170	Yes/No* Yes/No*	Heavy Very heavy
Digester Recirculation Underflow	 3 - 10 3 - 10	 0 - 1.5 0 - 6.0	 2.4 - 3.6 15 - 30	 No Yes/No*	Medium Very heavy
Chemically produced sludges: Alum/ferric - primary Lime – primary Lime – secondary	 0.5 - 310.0 1.0 - 6.0 2.0 - 15.0	 3 - 12 3 - 12 3 - 12	 9 - 20 9 - 25 9 - 25	 No No No	 Light Medium Medium
Incinerator slurries	0.5 - 10	0 - 15	6 - 30	Yes - High	Heavy

* Depends on de-gritting efficiency.

** High pressure for heat treatment.

*** TDH = Total Dynamic Head.

12.6.1.2 Capacity

Pump capacities should be adequate but not excessive. Provision for varying pump capacity is desirable.

12.6.1.3 Duplicate Units

Duplicate units should be provided where failure of one unit would seriously hamper plant operation.

12.6.1.4 Type

Plunger pumps, screw feed pumps, or other types of pumps with demonstrated solids handling capability should be provided for handling raw sludge. Where centrifugal pumps are used, a parallel positive displacement pump should be provided as an alternate to pump heavy sludge concentrations, such as primary or thickened sludge, that may exceed the pumping head of the centrifugal pump.

12.6.1.5 Minimum Head

A minimum positive head of 600 mm should be provided at the suction side of centrifugal type pumps and is desirable for all types of sludge pumps. Maximum suction lifts should not exceed 3 m for plunger pumps.

12.6.1.6 Head Loss

Figure 12.1 shows the multiplication factor to apply to the friction losses for turbulent flow of clean water to calculate the friction losses for untreated primary and concentrated sludges. Use of Figure 12.1 will often provide sufficiently accurate results for design, especially at solids concentrations below 3%, however, as pipe length, percent TS and percent volatile solids increase, more elaborate methods may have to be used to calculate the friction losses with sufficient accuracy.

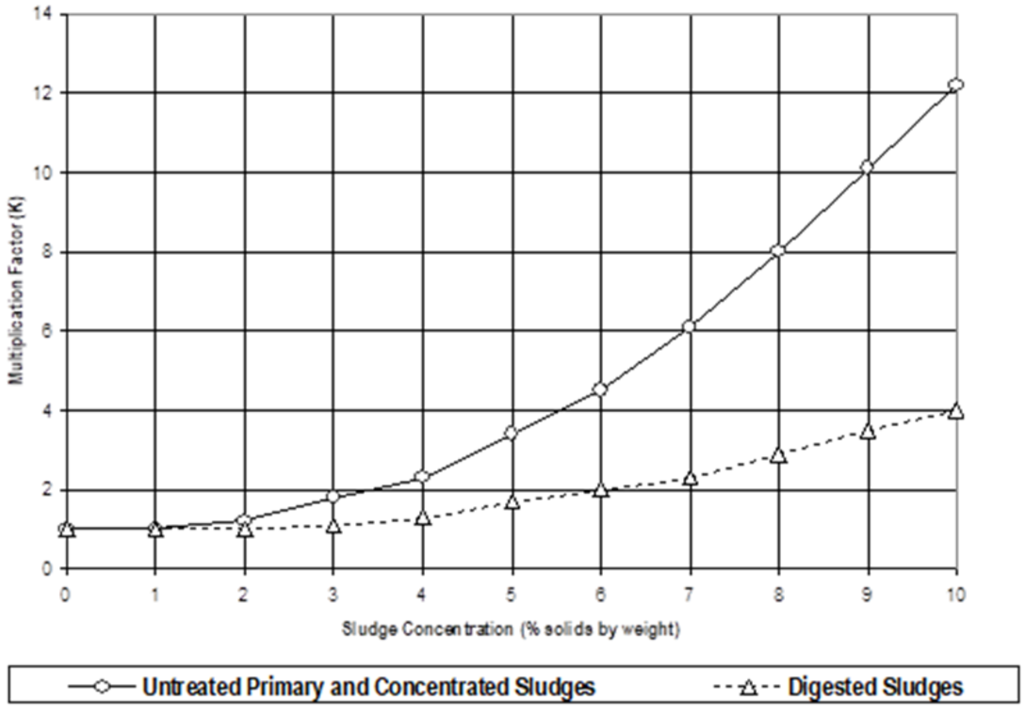


Figure 12.1: Approximate Friction Head Loss for Laminar Flow of Sludge

- Multiply loss with clean water by K to estimate friction loss under laminar conditions.
- The information on this figure has been extracted from *EPA 625/1-79-011 Process Design Manual for Sludge Treatment and Disposal* from the United States Environment Protection Agency.

12.6.1.7 Sampling Facilities

Unless sludge sampling facilities are otherwise provided, quick closing sampling valves should be installed at the sludge pumps. The size of valve and piping should be at least 40 mm and terminate at a suitable sized sampling sink or floor drain.

12.6.2 Sludge Piping

12.6.2.1 Size & Head

Sludge withdrawal piping should have a minimum diameter of 200 mm for gravity withdrawal and 150 mm for pump suction and discharge lines. Where withdrawal is by gravity, the available head on the discharge pipe should be adequate to provide at least 1.0 m/s velocity. Sludge pump velocities of 0.9 to 1.5 m/s should be developed. For heavier sludges and grease, velocities of 1.5 to 2.4 m/s are needed.

12.6.2.2 Slope

Gravity piping should be laid on uniform grade and alignment. The slope on gravity discharge piping should not be less than 3% for primary sludge and sludges thickened to greater than 2% solids. For aerobically digested or WASs (less than 2% solids), slope should not be less than 2%. Provisions should be made for draining and flushing discharge lines.

12.6.2.3 Supports

Special consideration should be given to the corrosion resistance and continuing stability of supporting systems for piping located inside the digestion tank.

12.7 Sludge Stabilization

12.7.1 Anaerobic Sludge Digestion

12.7.1.1 Applicability

Anaerobic digestion may be considered beneficial for sludge stabilization when the sludge volatile solids content is 50% or higher and if no inhibitory substances are present or expected. Anaerobic digestion of primary sludge is preferred over activated sludge because of the poor solids-liquid separation characteristics of activated sludges. Combining primary and secondary sludges will result in settling characteristics better than activated sludge but less desirable than primary alone. Chemical sludges containing lime, alum, iron, and other substances can be successfully digested if the volatile solids content remains high enough to support the biochemical reactions and no toxic compounds are present. If an examination of past sludge characteristics indicates wide variations in sludge quality, anaerobic digestion may not be feasible because of its inherent sensitivity to changing substrate quality. Table 12.8 lists sludges which are suitable for anaerobic digestion.

Table 12.8: Sludges Satisfactory for Anaerobic Digestion

Sludges Satisfactory for Anaerobic Digestion
Primary and lime.
Primary and ferric chloride.
Primary and alum.
Primary and trickling filter.
Primary, trickling filter, and alum.
Primary and waste activated.
Primary, waste activated, and lime.
Primary, waste activated, and alum.
Primary, waste activated, and ferric chloride.
Primary, waste activated, and sodium aluminate.

The advantages offered by anaerobic digestion include:

- Excess energy over that required by the process is produced. Methane is produced and can be used to heat and mix the reactor. Excess methane gas can be used to heat space, produce electricity, or as engine fuel.
- Quantity of TS for ultimate disposal is reduced. The volatile solids present are converted to methane, carbon dioxide, and water thereby reducing the quantity of solids. About 30 to 40% of the TS may be destroyed and 40 to 60% of the volatile solids may be destroyed.

- Product is a stabilized sludge that may be free from strong or foul odours and can be used for land application as ultimate disposal because the digested sludge contains plant nutrients.
- Pathogens are destroyed to a high degree during the process. Thermophilic digestion enhances the degree of pathogen destruction.
- Most organic substances found in municipal sludge are readily digestible except lignins, tannins, rubber, and plastics.

The disadvantages associated with anaerobic digestion include:

- Digester is easily upset by unusual conditions, including erratic or high loadings, and is very slow to recover.
- Operators must follow proper operating procedures.
- Heating and mixing equipment are required for satisfactory performance.
- Large reactors are required because of the slow growth of methanogens and required SRTs of 15 to 20 days for a high-rate system. Thus, capital cost is high.
- Resultant supernatant sidestream is a strong waste stream that greatly adds to the loading of the wastewater plant. It contains high concentrations of BOD, COD, SS, and ammonia nitrogen.
- Cleaning operations are difficult because of the closed vessel. Internal heating and mixing equipment can become major problems because of corrosion and wear in harsh inaccessible environments.
- Poor in dewatering characteristics.
- Possibility of explosion because of inadequate O&M, leaks, or operator carelessness exists.
- Gas line condensation or clogging can cause major maintenance problems.

12.7.1.2 Digestion Tanks & Number of Stages

With anaerobic sludge digestion facilities, the need for multiple units can often be avoided by providing 2-stage digestion along with sufficient flexibility in sludge pump and mixing so that 1-stage can be serviced while the other stage receives the raw sludge. Single stage digesters will generally not be satisfactory due to the usual need for sludge storage, and effective supernatant depth. They will be considered, however, where the designer can show that the above concerns can be satisfied, and that alternate means of sludge processing or emergency storage can be used in the event of breakdown.

12.7.1.3 Access Manholes

At least two 760 mm diameter access manholes should be provided in the top of the tank in addition to the gas dome. There should be stairways to reach the access manholes. A separate sidewall manhole should be provided. The opening should be large enough to permit the use of mechanical equipment to remove grit and sand. This manhole should be located near the bottom of the sidewall. All manholes should be provided with gas-tight and water-tight covers.

12.7.1.4 Safety

Non-sparking tools, safety lights, rubber-soled shoes, safety harness, gas detectors for inflammable and toxic gases and at least two self-contained breathing units should be provided for workers involved in cleaning the digesters.

Necessary safety facilities should be included where sludge gas is produced. All tank covers should be provided with pressure and vacuum relief valves and flame traps together with automatic safety shut-off valves. Water seal equipment should not be installed.

12.7.1.5 Field Data

Wherever possible, such as in the case of plant expansions, actual sludge quantity data should be considered for digester design. Often, due to errors introduced by poor sampling techniques, inaccurate flow measurements or unmeasured sludge flow streams, the sludge data from existing plants may be unsuitable for use in design, therefore, before sludge data is used for design, it should be assessed for its accuracy.

12.7.1.6 Typical Sludge Qualities & Generation Rates for Different Unit Processes

When reliable data are not available, the sludge generation rates and characteristics provided in Table 12.9 may be used.

Table 12.9: Typical Sludge Qualities & Generation Rates for Different Unit Processes

Unit Process	Liquid Sludge	Solids Concentration		Volatile Solids	Dry Solids	
	(L/m^3)	Range (%)	Average (%)	(%)	(g/m^3)	(g/cap·d)
Primary sedimentation with anaerobic digestion						
Undigested (no P removal)	2.0	(3.5 - 8)	5.0	65	120	55
Undigested (with P removal)	3.2	(3.5 - 7)	4.5	65	170	77
Digested (No P removal)	1.1	(5 - 13)	6.0	50	75	34
Digested (with P removal)	1.6	(5 - 13)	5.0	50	110	50
Primary sedimentation and conventional activated sludge with anaerobic digestion						
Undigested (no P removal)	4.0	(2 - 7)	4.5	65	180	82
Undigested (with P removal)	5.0	(2 - 6.5)	4.0	60	220	100
Digested (no P removal)	2.0	(2 - 6)	5.0	50	115	52
Digested (with P removal)	3.5	(2 - 6)	4.0	45	150	68
Contact stabilization and high rate with aerobic digestion						
Undigested (no P removal)	15.5	(0.4 - 2.8)	1.1	70	170	77
Undigested (with P removal)	19.1	(0.4 - 2.8)	1.1	60	210	95
Digested (no P removal)	6.1	(1 - 3)	1.9	70	115	52
Digested (with P removal)	8.1	(1 - 3)	1.9	60	155	70
Extended aeration with aerated sludge holding tank						
Waste activated (no P removal)	10.0	(0.4 - 1.9)	0.9	70	90	41
Waste activated (with P removal)	13.3	(0.4 - 1.9)	0.9	60	120	55
Sludge holding Tank (no P removal)	4.0	(0.4 - 5.0)	2.0	70	80	36
Sludge holding tank (with P removal)	5.5	(0.4 - 4.5)	2.0	60	110	50

Note:

- (L/cu. m) denotes litres of liquid sludge per m^3 of treated wastewater.
- (g/cu. m) denotes grams of dry solids per m^3 of treated wastewater.
- The above values are based on typical raw wastewater with:
  - Total BOD = 170 mg/L.
  - Soluble BOD = 50%.
  - SS = 200 mg/L.
  - Phosphorous (P) = 7 mg/L.
  - NH_4 = 20 mg/L.

12.7.1.7 Solids Retention Time

The minimum SRT for a low-rate digester should be 30 days. The minimum SRT of a high-rate digester should be 15 days.

12.7.1.8 Design of Tank Elements

12.7.1.8.1 Digester Shape

Anaerobic digesters are generally cylindrical in shape with inverted conical bottoms. Heat loss from digesters can be minimized by choosing a proper depth-diameter ratio such that the total surface area is the least for a given volume. A cylinder with diameter equal to depth can be shown to be the most economical shape from a heat loss viewpoint, however, structural requirements and scum control aspects also govern the optimum depth-diameter ratio.

12.7.1.9 Floor Shape

To facilitate draining, cleaning and maintenance, the following features are desirable:

The tank bottom should slope to drain toward the withdrawal pipe. For tanks equipped with mechanisms for withdrawal of sludge, a bottom slope not less than 1-to-12 (vertical-to-horizontal) is recommended. Where the sludge is to be removed by gravity alone, 3-to-12 slope is recommended.

12.7.1.10 Depth & Freeboard

For those units proposed to serve as supernatant development tanks, the depth should be sufficient to allow for the formation of a reasonable depth of supernatant liquor. A minimum water depth of 6 m is recommended. The acceptable range for sidewater depth is between 6 and 14 m.

The freeboard provided must take into consideration the type of cover and maximum gas pressure. For floating covers, the normal working water level in the tank under gas pressure is approximately 0.8 m below the top of the wall, thus providing from 0.5 to 0.6 m of freeboard between the liquid level and the top of the tank wall. For fixed flat slab roofs, a freeboard of 0.3 to 0.6 m above the working liquid level is commonly provided. For fixed conical or domed roofs, the freeboard between the working liquid level and the top of the wall inside the tank can be reduced to less than 0.3 m.

12.7.1.11 Scum Control

Scum accumulation can be controlled by including any of the following provisions in the equipment design:

a. Floating covers keep the scum layer submerged and thus moist and more likely to be broken up.
b. Discharging recirculated sludge on the scum mat serves the same purpose as (a).
c. Recirculating sludge gas under pressure through the tank liquors and scum.
d. Mechanically destroying the scum by employing rotating arms or a propeller in a draft tube.
e. A large depth-area ratio.
f. Or, a concentrated sludge feed to the digester.

Items (e) and (f) would release large volumes of gas per unit area, keep the scum in motion and mix the solids in the digester.

12.7.1.12 Grit & Sand Control

The digesters should be designed to minimize sedimentation of the particles and facilitate removal if settling takes place. These objectives can be achieved if tank contents are kept moving at 0.23 to 0.3 m/s and the floor slopes are about 1-to-4.

12.7.1.13 Alkalinity & pH Control

The effective pH range for methane producers is approximately 6.5 to 7.5 with an optimum range of 6.8 to 7.2. Maintenance of this optimum range is important to ensure good gas production and to eliminate digester upsets.

The stability of the digestion process depends on the buffering capacity of the digester contents; the ability of the digester contents to resist pH changes. The alkalinity is a measure of the buffering capacity of a freshwater system. Higher alkalinity values indicate a greater capacity for resisting pH changes. The alkalinity should be measured as bicarbonate alkalinity. Values for alkalinity in anaerobic digesters range from 1,500 to 5,000 mg/L as $CaCO_3$. The volatile acids produced by the acid producers tend to depress pH. Volatile acid concentrations under stable conditions range from 100 to 500 mg/L, therefore, a constant ratio below 0.25 of volatile acids to alkalinity should be maintained so that the buffering capacity of the system can be maintained.

Sodium bicarbonate, lime, sodium carbonate, and ammonium hydroxide application are recommended for increasing alkalinity of digester contents.

12.7.1.14 Mixing

Thorough mixing via digester gas (compressor power requirement 5 to 8 W/m^3) or mechanical means (6.6 W/m^3) in the primary stage will be necessary in all cases when digesters are proposed. This mixing should assure the homogeneity of the digester contents and prevent stratification. Actual power requirements should be based on tank size, geometry, sludge rheology, type of mixer, and mixing energy or shear rate required.

Gas mixing systems recirculate compressed digester gas in either unconfined or confined mixing; both creating upward mixing action.

Mechanical mixing uses axial flow propellers with roof or external mounted draft tubes. The roof mounted draft tubes limit the digester size to less than 24 m, while external mounted tubes can accommodate diameters greater than 24 m. Mechanical mixing utilizing vertical mixing action may also be considered.

Pump mixing uses axial flow patterns, and screw type centrifugal or chopper type pumps. These systems withdraw sludge from the bottom and pump it back to higher elevation within the digester. Where sludge recirculation is employed, pumps and associated pipework should meet the general intent of Section 12.6.

One concern with mixing is the formation of foam and grease on the surface of the digester. Means of suppression and/or removal should be considered.

12.7.1.15 Sludge Inlets, Outlets, Recirculation, & High-Level Overflow

12.7.1.15.1 Multiple Inlets & Draw-Offs

Multiple sludge inlets and draw-offs and, where used, multiple recirculation suction and discharge points to facilitate flexible operation and effective mixing of the digester contents, should be provided unless adequate mixing facilities are provided within the digester.

12.7.1.15.2 Inlet Configurations

One inlet should discharge above the liquid level and be located at approximately the center of the tank to assist in scum breakup. The second inlet should be opposite to the suction line at approximately the 0.7 diameter point across the digester.

12.7.1.15.3 Inlet Discharge Location

Raw sludge inlet discharge points should be so located as to minimize short circuiting to the digested sludge or supernatant draw-offs.

12.7.1.15.4 Sludge Withdrawal

Sludge withdrawal to disposal should be from the bottom of the tank. The bottom withdrawal pipe should be interconnected with the necessary valving to the recirculation piping to increase operational flexibility when mixing the tank contents.

12.7.1.15.5 Emergency Overflow

An un-valved vented overflow should be provided to prevent damage to the digestion tank and cover in case of accidental overfilling. This emergency overflow should be piped to an appropriate point and at an appropriate rate in the treatment process or side stream treatment plant to minimize the impact on process units.

12.7.1.16 Primary Tank Capacity

The primary digestion tank capacity should be determined by rational calculations based upon such factors as volume of sludge added, percent solids and character, the temperature to be maintained in the digesters, the degree or extent of mixing to be obtained; and the degree of volatile solids reduction required. Calculations should be submitted to justify the basis of design.

When such calculations are not based on the above factors, the minimum primary digestion tank capacity outlined in Section 12.5.5.9 will be required. Such requirements assume that the raw sludge is derived from ordinary domestic wastewater, a digestion temperature is to be maintained in the range of 32 to 39°C, that 40 to 50% volatile matter will be maintained in the digested sludge, and that the digested sludge will be removed frequently from the system.

12.7.1.16.1 High-Rate Digester

The primary high-rate digester should provide for intimate and effective mixing to prevent stratification and to assure homogeneity of digester content. The system may be loaded at a rate up to 1.3 kg of volatile solids per m^3 of volume per day in the active digestion unit. When grit removal facilities are not provided, the reduction of digester volume due to grit accumulation should be considered.

12.7.1.16.2 Low-Rate Digester

For low-rate digesters where mixing is accomplished only by circulating sludge through an external heat exchanger, the system may be loaded up to 0.64 kg of volatile solids per m^3 of volume per day in the active digestion unit. This loading may be modified upward or downward depending upon the degree of mixing provided.

12.7.1.17 Secondary Digester Sizing

The secondary digester should be sized to permit solids settling for decanting and solids thickening operations, and in conjunction with possible off-site facilities, to provide the necessary digested sludge storage. The necessary total storage time will depend upon the means of ultimate sludge disposal, with the greatest time required with soil conditioning operations (winter storage), and with less storage required with landfilling or incineration as ultimate disposal methods. Offsite storage in sludge lagoons, sludge storage tanks, or other facilities may be used to supplement the storage capacity of the secondary digester. If high-rate primary digesters are used and efficient dewatering within the secondary digester is required, the secondary digester must be conservatively sized to allow adequate solids separation (secondary to primary sizing ratios of 2-to-1 to 4-to-1 are recommended).

12.7.1.18 Digester Covers

To provide gas storage volume and to maintain uniform gas pressures, a separate gas storage sphere should be provided, or at least one digester cover should be of the gas-holder floating type. If only one floating cover is provided, it should be on the secondary digester. Insulated pressure and vacuum relief valves and flame traps should be provided. Access manholes and at least two 200 mm sampling wells should also be provided on the digester covers.

Steel is the most commonly used material for digester covers, however, other properly designed and constructed materials have also been successfully employed, such as concrete, fibreglass, and membrane.

12.7.1.19 Sludge Piping

Maximum flexibility should be provided in terms of sludge transfer from primary and secondary treatment units to the digesters, between the primary and secondary digesters, and from the digesters to subsequent sludge handling operations. The minimum diameter of sludge pipes should be 200 mm for gravity withdrawal and 150 mm for pump suction and discharge lines. Provision should be made for flushing and cleaning sludge piping. Sampling points should be provided on all sludge lines. Main sludge transfer lines should be from the bottom of the primary digester to the mid-point of the secondary digester. Additional transfer lines should be from intermediate points in the primary digester (these can be dual-purpose supernatant and sludge lines).

12.7.1.20 Overflows

Each digester should be equipped with an emergency overflow system.

12.7.1.21 Gas Collection, Piping, & Appurtenances

12.7.1.21.1 General

All portions of the gas system including the space above the tank liquor, storage facilities and piping should be so designed that under all normal operating conditions, including sludge withdrawal, the gas will be maintained under positive pressure. All enclosed areas where any gas leakage might occur should be adequately ventilated.

All gas collection equipment, piping and appurtenances should comply with the *CSA/ANSI B149.6 Code for digester gas, landfill gas, and biogas generation and utilization*.

12.7.1.21.2 Safety Equipment

All necessary safety facilities should be included where gas is produced. Pressure and vacuum relief valves and flame traps together with automatic safety shut-off valves, are essential, and should be protected from freezing. Water seal equipment should not be installed. Gas safety equipment and gas compressors should be housed in a separate room with an exterior entrance.

Provision should also be made for automatically purging the combustion chamber of the heating unit thoroughly with air after a shut-down or pilot light failure, and before it can be ignited. This will provide certainty that no explosive mixture exists within the unit.

12.7.1.21.3 Gas Piping & Condensate

The main gas collector line from the digestion tanks should be at least 100 mm in diameter with the gas intake being well above the digester scum level, generally at least 1.2 m above the maximum liquid level in the tank. A smaller diameter pipe at the gas production meter is acceptable. If gas mixing is used, the gas withdrawal pipe must be of sufficient size to limit the pressure drop in terms of the total gas flow from the digester. Such flow includes not only the daily gas production, but also the daily gas recycling flow. The recycling gas flow information should be combined with the estimated peak daily gas flow data to determine the proper piping size.

Gas pipe slopes of 20 mm/m are desirable with a minimum slope of 10 mm/m for drainage. The maximum velocity in sludge-gas piping should be limited to not more than 3.4 or 3.7 m/s.

Gas piping should slope to condensation traps at low points in a location not subject to freezing. The use of float-controlled condensate traps is not permitted.

Adequate pipe support is essential to prevent breaking, and special care should be given where pipes are located underground.

Gas piping and pressure relief valves must include adequate flame traps. They should be installed as close as possible to the device serving as a source of ignition.

12.7.1.21.4 Gas Utilization Equipment

Gas burning boilers, engines, etc., should be located at ground level and in well ventilated rooms, not connected to the digester gallery. Gas lines to these units should be provided with suitable flame traps.

12.7.1.21.5 Electrical Systems

Electrical equipment, fixtures and controls, in places enclosing anaerobic digestion appurtenances, where hazardous may accumulate, should comply with the latest edition of *NFPA 820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities*. Digester galleries should be isolated from normal operating areas.

12.7.1.21.6 Waste Gas

Waste gas burners should be readily accessible and should be located at least 15 m away from any plant structure if placed at ground level. Waste gas burners should be of sufficient height and so located to prevent injury to personnel due to wind or downdraft conditions.

All waste gas burners should be equipped with automatic ignition, such as a pilot light or a device using a photoelectric cell sensor. Consideration should be given to the use of natural or propane gas to insure reliability of the pilot light.

Provision for condensate removal, pressure control, and flame protection ahead of waste burners is required.

12.7.1.21.7 Ventilation

Any underground enclosures connecting with digestion tanks or containing sludge or gas piping or equipment should be provided with forced ventilation in accordance with the latest edition of *CAN/CSA-B105-M93 Code for Digester Gas and Landfill Gas Installations* and *NFPA 820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities*. Tightly fitting self-closing doors should be provided at connecting passageways and tunnels to minimize the spread of gas.

12.7.1.21.8 Meter

A gas meter with bypass should be provided, to meter total gas production for each active digestion unit. Total gas production for 2-stage digestion systems operated in series may be measured by a single gas meter with proper interconnected gas piping.

Where multiple primary digestion units are utilized with a single secondary digestion unit, a gas meter should be provided for each primary digestion unit. The secondary digestion unit may be interconnected with the gas measurement unit of one of the primary units. Interconnected gas piping should be properly valved with gas tight valves to allow measurement of gas production from either digestion unit or maintenance of either digestion unit.

Gas meters may be of the orifice plate, turbine, thermal, or vortex type. Positive displacement meters should not be utilized. The meter must be specifically designed for contact with corrosive and dirty gases.

12.7.2 Digestion Tank Heating

12.7.2.1 Heating Capacity

12.7.2.1.1 Capacity

Sufficient heating capacity should be provided to consistently maintain the design sludge temperature considering insulation provisions and ambient cold weather conditions. Where digestion tank gas is used for other purposes, an auxiliary fuel may be required. The design operating temperature should be in the range of 29 to 38°C where optimum mesophilic digestion is required.

12.7.2.2 Insulation

Wherever possible, digestion tanks should be constructed above ground-water level and should be suitably insulated to minimize heat loss. Maximum utilization of earthen bank insulation should be used.

12.7.2.3 Heating Facilities

Sludge may be heated by circulating the sludge through external heaters or by heating units located inside the digestion tank. The external heat exchanger systems are preferred.

12.7.2.3.1 External Heating

Piping should be designed to provide for the preheating of feed sludge before introduction to the digesters. Provisions should be made in the lay-out of the piping and valving to facilitate heat exchanger tube removal and cleaning of these lines. Heat exchanger sludge piping should be sized for peak heat transfer requirements. Heat exchangers should have a heating capacity of 130% of the calculated peak heating requirement to account for the occurrence of sludge tube fouling.

12.7.2.3.2 Other Heating Methods

The use of hot water heating coils affixed to the walls of the digester, or other types of internal heating equipment that require emptying the digester contents for repair, are not acceptable.

Other systems and devices have been developed recently to provide both mixing and heating of anaerobic digester contents. These systems will be reviewed on their own merits. Operating data detailing their reliability and O&M characteristics will be required.

12.7.2.4 Hot Water Internal Heating Controls

12.7.2.4.1 Mixing Valves

A suitable automatic mixing valve should be provided to temper the boiler water with return water so that the inlet water to the removable heat jacket or coils in the digester can be held below a temperature at which caking will be accentuated. Manual control should also be provided by suitable by-pass valves.

12.7.2.4.2 Boiler Controls

The boiler should be provided with suitable automatic controls to maintain the boiler temperature at approximately 82°C to minimize corrosion and to shut off the main gas supply in the event of pilot burner or electrical failure, low boiler water level, ow gas pressure, or excessive boiler water temperature or pressure.

12.7.2.4.3 Boiler Water Pumps

Boiler water pumps should be sealed and sized to meet the operating conditions of temperature, operating head, and flowrate. Duplicate units should be provided.

12.7.2.4.4 Thermometers

Thermometers should be provided to show inlet and outlet temperatures of sludge, hot water feed, hot water return, and boiler water.

12.7.2.4.5 Water Supply

The chemical quality should be checked for suitability for this use.

12.7.2.5 External Heater Operating Controls

All controls necessary to insure effective and safe operation are required. Provision for duplicate units in critical elements should be considered.

12.7.2.6 Supernatant Withdrawal

12.7.2.6.1 Piping Size

Supernatant piping should not be less than 150 mm in diameter. Precaution must be taken to avoid loss of digester gas through supernatant piping.

12.7.2.7 Withdrawal Arrangement

12.7.2.7.1 Withdrawal Levels

Piping should be arranged so that withdrawal can be made from three or more levels in the tank. A positive un-valved vented overflow should be provided. The emergency overflow should be piped to an appropriate point and at an appropriate rate in the treatment process or side stream treatment unit process to minimize the impact.

Both primary and secondary digesters should be equipped with supernating lines, so that during emergencies the primary digester can be operated as a single stage process.

12.7.2.7.2 Supernatant Selector

A fixed screen supernatant selector or similar type device should be limited for use in an unmixed secondary digestion unit If a supernatant selector is provided, provisions should be made for at least one other draw-off level located in the supernatant zone of the tank, in addition to the unvalved emergency supernatant draw-off pipe. High pressure back-wash facilities should be provided.

12.7.2.7.3 Withdrawal Selection

On fixed cover tanks the supernatant withdrawal level should preferably be selected by means of interchangeable extensions at the discharge end of the piping.

12.7.2.8 Sampling

Provision should be made for sampling at each supernatant draw-off level. Sampling pipes should be at least 40 mm in diameter and should terminate at a suitably-sized sampling sink or basin.

12.7.2.9 Alternate Supernatant Disposal

An alternate disposal method for the supernatant liquor such as a lagoon, an additional sand bed or hauling from the plant site should be provided for use in case supernatant is not suitable or other conditions make it advisable not to return it to the plant. Consideration should be given to supernatant conditioning where appropriate in relation to its effect on plant performance and effluent quality.

12.7.2.10 Sludge Sampling Requirements

An adequate number of sampling pipes at proper locations should enable the operator to assess the quality of the contents and to know how much sludge is in the digesters. The following requirements should govern the design:

- To avoid clogging, sludge sampling pipes should be at least 75 mm in diameter.
- Provision should be made for the connection of a water source of adequate pressure to these pipes for back flushing when the need arises.
- There should be at least three sampling pipes each separately valved for the primary digesters and four for the secondary digesters.

12.7.3 Aerobic Sludge Digestion

Aerobic digestion is accomplished in single or multiple tanks, designed to provide effective air mixing, reduction of the organic matter, supernatant separation, and sludge concentration under controlled conditions.

12.7.3.1 Applicability

Aerobic digestion is considered suitable for secondary sludge or a combination of primary and secondary sludges. Table 12.10 provides the advantages and disadvantages in the use of aerobic sludge digestion.

Table 12.10: Advantages & Disadvantages of Aerobic Sludge Digestion

Advantages	Disadvantages
Low initial cost particularly for small plants.	High energy costs.
Supernatant less objectionable than anaerobic.	Generally lower Volatile Suspended Solids (VSS) destruction than anaerobic.
Simple operational control.	Reduced pH and alkalinity.
Broad applicability.	Potential for pathogen spread through aerosol drift.
If properly designed, does not generate nuisance odours.	Sludge is typically difficult to dewater by mechanical means.
Reduces total sludge mass.	Cold temperatures adversely affect performance.

12.7.3.2 Field Data

Wherever possible, such as in the case of plant expansions, actual sludge quantity data should be considered for digester design. Often, due to errors introduced by poor sampling techniques, inaccurate flow measurements or unmeasured sludge flow streams, the sludge data from existing plants may be unsuitable for use in design. Before sludge data is used for design, it should be assessed for its accuracy.

12.7.3.3 Multiple Units

Multiple digestion units capable of independent operation are desirable and should be provided in all plants where the design average flow exceeds 380 m^3/d. All plants not having multiple units should provide alternate sludge handling and disposal methods.

12.7.3.4 Pre-Treatment

Thickening of sludge is recommended prior to aerobic digestion.

12.7.3.5 Design Considerations

Factors which should be considered when designing aerobic digesters include:

- Type of sludge to be digested.
- Ultimate method of disposal.
- Required winter storage.
- Digester pH.
- Sludge temperature.
- Raw sludge quality.

12.7.3.6 Solids Retention Time

A minimum SRT of 45 days is required. If local conditions require a more stable sludge, a sludge age of 90 days should be necessary. To produce a completely stable sludge, a sludge age in excess of 120 days is required.

12.7.3.7 Hydraulic Retention Time

The minimum required HRT for aerobic digesters provided with pre-thickening facilities are as follows:

Table 12.11: Minimum Hydraulic Retention Time

Minimum HRT (days)	Type of Sludge
25	WAS only
25	Trickling filter sludge only
30	Primary plus secondary sludge

The more critical of the two guidelines, SRT and HRT, should govern the design.

12.7.3.8 Tank Capacity

The determination of tank capacities should be based on rational calculations, including such factors as quantity of sludge produced, sludge characteristics, time of aeration and sludge temperature.

Calculations should be submitted to justify the basis of design.

When such calculations are not based on the above factors, the minimum combined digestion tank capacity should be based on the following:

- Volatile solids loading should not exceed 1.60 kg/m^3·d in the digestion units.
- Lower loading rates may be necessary depending on temperature, type of sludge, and other factors.

If 45 days solid retention time is all that is provided, it is suggested that 2/3 of the total digester volume be in the first tank and 1/3 be in the second tank. Actual storage requirements will depend upon the ultimate disposal operation. Any minor additional storage requirements may be made up in the second stage digester, but if major additional storage volumes are required, separate on-site or off-site sludge storage facilities should be considered to avoid the power requirements associated with aerating greatly oversized aerobic digesters.

12.7.3.9 Air & Mixing Requirements

Aerobic sludge digestion tanks should be designed for effective mixing by satisfactory aeration equipment. Sufficient air should be provided to keep the solids in suspension and maintain DO from 1.0 to 2.0 mg/L. A minimum mixing and air requirement of 0.50 L/s/m^3 of tank volume should be provided with the largest blower out of service. If diffusers are used, the non-clog type is recommended, and they should be designed to permit continuity of service. Air supply to each tank should be valved separately to allow aeration shut-down in either tank. All diffuser drop pipes should be able to withstand impact of ice masses that may form in winter and should allow for easy removal for maintenance. If mechanical aerators are utilized, at least two turbine aerators per tank should be provided. Use of mechanical equipment is discouraged where freezing temperatures are normally expected.

12.7.3.10 Tank Configuration

Aerobic digesters are generally open tanks. The tankage should be of common wall construction or earthen-bermed to minimize heat loss. Tank depths should be between 3.5 to 4.5 m; tanks and piping should be designed to permit sludge addition, sludge withdrawal, and a supernatant decanting zone from various depths.

Freeboard depths of at least 0.9 to 1.2 m should be provided to account for excessive foam levels. Floor slopes of 1-to-12 to 3-to-12 should be provided.

12.7.3.11 Supernatant Separation, Scum, & Grease Removal

12.7.3.11.1 Supernatant Separation

Facilities should be provided for effective formation of a good quality supernatant. Separate facilities are recommended, however, supernatant separation may be accomplished in the digestion tank provided additional volume is provided. The supernatant draw-off unit should be designed to prevent recycle of scum and grease back to plant process units. Provisions should be made to withdraw supernatant from multiple levels of the supernatant withdrawal zone.

12.7.3.11.2 Scum & Grease Removal

Facilities should be provided for the effective collection of scum and grease from the aerobic digester for final disposal and to prevent its recycle back to the plant process, and to prevent long term accumulation and potential discharge in the effluent.

12.7.3.12 High-Level Emergency Overflow

An un-valved high-level overflow and any necessary piping should be provided to return digester overflow back to the head of the plant or to the aeration process in case of accidental overfilling. Design considerations related to the digester overflow should include waste sludge rate and duration during the period the plant is unattended, potential effects on plant process units, discharge location of the emergency overflow, and potential discharge or SS in the plant effluent.

12.7.3.13 Digested Sludge Storage Volume

12.7.3.13.1 Sludge Storage Volume

Sludge storage must be provided in accordance with Section 12.5.2 to accommodate daily sludge production volumes and as an operational buffer for unit outage and adverse weather conditions. Designs utilizing increased sludge age in the activated sludge system as a means of storage are not acceptable.

12.7.3.13.2 Liquid Sludge Storage

Liquid sludge storage facilities should be based on the values in Table 12.12 unless digested sludge thickening facilities are utilized to provide solids concentrations of greater than 2%.

Table 12.12: Sludge Source

Sludge Source	Volume (m^3/PE day)
WAS – no primary settling, primary plus WAS.	0.004
WAS – exclusive of primary sludge.	0.002
Primary plus fixed film reactor sludge.	0.003

12.7.4 High pH Stablization

12.7.4.1 General

Alkaline material may be added to liquid primary or secondary sludges for sludge stabilization in lieu of digestion facilities, to supplement existing digestion facilities, or for interim sludge handling. There is no direct reduction of organic matter or sludge solids with the high pH stabilization process. There is an increase in the mass of dry sludge solids. Without supplemental dewatering, additional volumes of sludge will be generated. The design should account for the increased sludge quantities for storage, handling, transportation, and disposal methods and associated costs.

12.7.4.2 Operational Criteria

Sufficient alkaline material should be added to liquid sludge in order to produce a homogeneous mixture with a minimum pH of 12 after 2 hours of vigorous mixing. Facilities for adding supplemental alkaline material should be provided to maintain the pH of the sludge during interim sludge storage periods.

12.7.4.3 Odour Control & Ventilation

Odour control facilities should be provided for sludge mixing and treated sludge storage tanks when located within 800 m of residential or commercial areas. The Regulatory Authority should be contacted for design and air pollution control objectives to be met for various types of air scrubber units. Ventilation is required for indoor sludge mixing, storage, or processing facilities. Ventilation requirements must be consistent with the area classification as determined by the latest edition of *NFPA 820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities*.

12.7.4.4 Mixing Tanks & Equipment

12.7.4.4.1 Tanks

Mixing Tanks may be designed to operate as either a batch or continuous flow process. A minimum of two tanks should be provided of adequate size to provide a minimum 2 hours CT in each tank. The following items should be considered in determining the number and size of tanks:

- Peak sludge flowrates.
- Storage between batches.
- Dewatering or thickening performed in tanks.
- Repeating sludge treatment due to pH decay of stored sludge.
- Sludge thickening prior to sludge treatment.
- Type of mixing device used and associated maintenance or repair requirements.

12.7.4.4.2 Equipment

Mixing equipment should be designed to provide vigorous agitation within the mixing tank, maintain solids in suspension, and provide for a homogeneous mixture of the sludge solids and alkaline material. Mixing may be accomplished either by diffused air or mechanical mixers. If diffused aeration is used, an air supply of 0.50 L/m^3s of mixing tank volume should be provided with the largest blower out of service. When diffusers are used, the non-clog type is recommended, and they should be designed to permit continuity of service. If mechanical mixers are used, the impellers should be designed to minimize fouling with debris in the sludge and consideration should be made to provide continuity of service during freezing weather conditions.

12.7.4.5 Chemical Feed & Storage Equipment

12.7.4.5.1 General

Alkaline material is caustic in nature and can cause eye and tissue injury. Equipment for handling or storing alkaline material should be designed for adequate operator safety. Storage, slaking, and feed equipment should be sealed as airtight as practical to prevent contact of alkaline material with atmospheric carbon dioxide and water vapour and to prevent the escape of dust material. All equipment and associated transfer lines or piping should be accessible for cleaning.

12.7.4.5.2 Feed & Slaking Equipment

The design of the feeding equipment should be determined by the treatment plant size, type of alkaline material used, slaking required, and operator requirements. Equipment may be either of batch or automated type. Automated feeders may be of the volumetric or gravimetric type depending on accuracy, reliability, and maintenance requirements. Manually operated batch slaking of quicklime (CaO) should be avoided unless adequate protective clothing and equipment are provided. At small plants, used of hydrated lime ($Ca(OH)_2$) is recommended over quicklime due to safety and labour-saving reasons. Feed and slaking equipment should be sized to handle a minimum of 150% of the peak sludge flowrate including sludge that may need to be retreated due to pH decay. Duplicate units should be provided.

12.7.4.5.3 Chemical Storage Facilities

Alkaline materials may be delivered either in bag or bulk form depending upon the amount of material used. Material delivered in bags must be stored indoors and elevated above floor level. Bags should be of the multi-wall moisture-proof type. Dry bulk storage containers must be as airtight as practical and should contain a mechanical agitation mechanism. Storage facilities should be sized to provide a minimum of a 30-day supply.

12.7.4.6 Sludge Storage

Refer to Section 12.5.2 for general design considerations for sludge storage facilities.

The design should incorporate the following considerations for the storage of high pH stabilized sludge:

- Liquid sludge
- Dewatered sludge
- Off-site storage

12.7.4.6.1 Liquid Sludge

Liquid high pH stabilized sludge should not be stored in a lagoon. Said sludge should be stored in a tank or vessel equipped with rapid sludge withdrawal mechanisms for sludge disposal or retreatment. Provisions should be made for adding alkaline material in the storage tank. Mixing equipment should be provided in all storage tanks.

12.7.4.6.2 Dewatered Sludge

On-site storage of dewatered high pH stabilized sludge should be limited to 30 days. Provisions for rapid retreatment or disposal of dewatered sludge stored on-site should also be made in case of sludge pH decay. Dewatered sludge cake should be suitably protected from weather for long term storage applications.

12.7.4.6.3 Off-Site Storage

There should be no off-site storage of high pH stabilized sludge unless specifically permitted by the Regulatory Authority.

12.7.4.7 Disposal

Immediate sludge disposal methods and options are recommended to be utilized to reduce the sludge inventory on the treatment plant site and amount of sludge that may need to be retreated to prevent odours if sludge pH decay occurs.

12.8 Advanced Treatment Alternatives for Pathogen Reduction

Since *40 CFR-Part 503* in the *Standards for the Use or Disposal of Sewage Sludge* from the United States Environment Protection Agency was published in 1993, stabilized sludge has been classified as Exceptional Quality (EQ), Class A, and Class B Bio-solids and the corresponding restrictions placed on their disposal and Processes to Further Reduce Pathogens (PFRPs) are being developed and marketed.

The purpose of this section will be to describe some of the sludge digestion methods and PFRPs that have become popular as a result of *Rule 503* in the *Standards for the Use or Disposal of Sewage Sludge* from the United States Environment Protection Agency. These methods may have application in Atlantic Canada wherever a need exists for a high-quality end product due to restrictions that may exist for final disposal.

12.8.1 Processes to Further Reduce Pathogens

Unstabilized sludge contains putrescible organic substances, as well as pathogenic forms of bacteria, viruses, worm eggs, and the like. Sludge treatment processes that are classified as PFRPs must reduce both the organics and pathogens to set levels. PFRP alternatives include composting, thermal drying, heat treatment, autothermal thermophilic aerobic digestion, irradiation, and pasteurization. The most common and applicable method in Atlantic Canada is composting, described below.

12.8.1.1 Composting

Composting is a process in which organic material undergoes biological degradation to a stable end product. Sludge that has been composted properly is a sanitary, nuisance-free, humus-like material.

The Canadian Food Inspection Agency (CFIA) regulates the use of composts being sold in accordance with the Fertilizers Regulations made under the *Fertilizers Act* from the Canadian Food Inspection Agency. *Guidelines for Compost Quality* from the Canadian Council of Ministers of the Environment specifies criteria for product safety and quality: foreign matter, maturity, pathogens, and trace elements. The *National Standard of Canada - Organic Soil Conditioners – Composts* from the Standards Council of Canada (SCC) provides voluntary guidelines.

The designer should be familiar with the current requirements of these regulations and guidelines and consult with the Regulatory Authority regarding site-specific design and operating criteria related to buffer zones, storage of composting material, runoff or leachate control, odour control and other process issues.

Composting is accomplished under aerobic conditions. Approximately 20 to 30% of the volatile solids are converted to carbon dioxide and water. As the organic material in the sludge decomposes, the compost heats to temperatures in the pasteurization range of 50 to 70°C, and enteric pathogenic organisms are destroyed. A properly composted sludge may be used as a soil conditioner in agricultural or horticultural applications or for final disposal, subject to any limitations based on constituents in the sludge.

Most composting operations consist of the following basic steps:

- Mixing dewatered sludge with an amendment and/or a bulking agent.
- Aerating the compost pile either by the addition of air, mechanical turning, or both.
- Recovery of the bulking agent (if practical).
- Further curing and storage.
- Final disposal.

An amendment is an organic material added to the feed substrate, primarily to reduce the bulk weight and increase the air voids for proper aeration. Amendments can also be used to increase the quantity of degradable organics in the mixture. Commonly used amendments are sawdust, straw, recycled compost, and rice hulls. A bulking agent is an organic or inorganic material used to provide structural support and to increase the porosity of the mixture for effective aeration. Wood chips are the most commonly used bulking agents and can be recovered and reused. Aeration is required not only to supply oxygen, but to control the composting temperature and remove excess moisture.

Three major types of composting systems used are the aerated static pile, windrow, and in-vessel (enclosed mechanical) systems.

12.8.1.1.1 Aerated Static Pile

The aerated static pile system consists of a grid of aeration or exhaust piping over which a mixture of dewatered sludge and bulking agent is placed. In a typical static pile system, the bulking agent consists of wood chips, which are mixed with the dewatered sludge by a pug mill type or rotating drum mixer or by movable equipment such as a front-end loader. Material is composted for 21 to 28 days and is typically cured for another 30 days of longer. Typical pile heights are about 2.0 to 2.5 m. A layer of screened compost is often placed on top of the pile for insulation. Perforated plastic drainage pipes are commonly used for air supply and each individual pile is recommended to have an individual blower for more effective aeration control. Screening of the cured compost is usually done to reduce the quantity of the end product requiring ultimate disposal and to recover the bulking agent. For improved process and odour control, many new plants cover or enclose all or significant portions of the system.

12.8.1.1.2 Windrow

In a windrow system, the mixing and screening operations are similar to those for the aerated static pile operation. Windrows are constructed 1.0 to 2.0 m high and 2.0 to 4.3 m at the base. The rows are turned and mixed periodically during the composting period. Supplemental mechanical aeration is used in some applications. Under typical operating conditions, the windrows are turned a minimum of five times while the temperature is maintained at or above 55°C. Turning of the windrows is often accompanied by the release of offensive odours. The composting period is about 21 to 28 days. In recent years, specialized equipment has been developed to mix the sludge and the bulking agent and to turn the composting windrows. Some windrow operations are covered or enclosed, similar to aerated static piles.

12.8.1.1.3 In-Vessel Composting Systems

In-vessel composting is accomplished inside an enclosed container or vessel. Mechanical systems are designed to minimize odours and process time by controlling conditions such as air flow, temperature, moisture, and oxygen concentration. The systems are compact and automated to control environmental conditions in an effort to reduce composting time. Air supplied by blowers is forced through the mixture which is periodically agitated. In-vessel designs include vertical plug flow, horizontal plug flow, and agitated bin reactors.

12.8.1.1.4 Design Considerations

The factors that must be considered in the design of a composting system are presented in Table 12.13.

Table 12.13: Design Considerations for Aerobic Sludge Composting Processes

Item	Comment
Type of sludge	Both untreated and digested sludge can be composted successfully. Untreated sludge has a greater potential for odors, particularly for windrow systems. Untreated sludge has more energy available, will degrade more readily, and has a higher oxygen demand.
Amendments and bulking agents	Amendment and bulking agent characteristics, such as moisture content, particle size, and available carbon, affect the process and quality of the product. Bulking agents should be readily available. Wood chips, sawdust, recycled compost, and straw have been used.
Carbon:Nitrogen (C:N) ratio	The initial C:N ratio should be in the range of 25:1 to 35:1 by weight. Carbon should be checked to ensure it is easily biodegradable.
Volatile solids	The volatile solids of the composting mix should be greater than 50%.
Air requirements	Air with at least 50% of the oxygen remaining should reach all parts of the composting material for optimum results, especially in mechanical systems.
Moisture content	Moisture content of the composting mixture should not be greater than 60% for static pile and windrow composting and not greater than 65% for in-vessel composting.
pH	pH of the composting mixture should generally be in the range of 6 to 9.
Temperature	The optimum temperature for biological stabilization is between 45 and 55°C. For best results, the temperature should be maintained between 50 and 55°C for the first few days and between 55 and 60°C for the remainder of the composting period. If the temperatures are allowed to increase beyond 60°C for a significant period of time, biological activity will be reduced.
Mixing and turning	To prevent drying, caking, and air channelling, material in the process of being composted should be mixed or turned on a regular schedule as required. Frequency of mixing or turning will depend on the type of composting operation.
Heavy metals and trace organics	Heavy metals and trace organics in the sludge and finished compost should be monitored to ensure that the concentrations do not exceed the applicable regulations for end use of the product.
Site constraints	Factors to be considered in selecting a site include available area, access, site drainage/runoff, proximity to treatment plant and other land uses, odour control, climatic conditions, and availability of buffer zone.
Climate change	Consider the site’s sensitivity and resilience to the impacts of climate change.

12.8.1.1.5 Co-Composting with Solid Wastes

Co-composting of sludge and municipal solid wastes may not require sludge dewatering. Feed sludges may have a solids content ranging from 5 to 12%. A 2-to-1 mixture of solid wastes to sludge is recommended as a minimum. The solid wastes should be pre-sorted and pulverized in a hammermill prior to mixing with sludge.

12.9 Sludge Recycling & Disposal Methods

When sludge recycling or disposal methods, such as utilization on land, land reclamation, incineration, lagoons and/or landfill are considered, pertinent requirements from the Regulatory Authority should be followed. Sludge quality will be a significant consideration in determine appropriate recycling or disposal options.

12.9.1 Sludge Utilization on Land

All jurisdictions support the beneficial use of biosolids. Land spreading of sludge may be a viable option depending upon the quality of the sludge and local conditions. Land application programs must also consider issues such as stabilization, storage, transportation, application, soil, crop, and groundwater, including climate change considerations such as impacts to groundwater recharge. Applications should be matched with the crop nutritional needs to avoid build up of phosphorous in soils, loss of nitrogen to groundwater. Some jurisdictions allow sludge to be utilized on land provided that both the sludge and disposal area meet the requirements listed in Appendix E. In New Brunswick, only sludge that has undergone further treatment to reduce pathogens as listed in Section E.1 and meets equivalent metal concentrations as those listed for Category A compost in the *Guidelines for Compost Quality* from the Canadian Council of Ministers of the Environment can be utilized on land.

System Owners considering land application should discuss their plans in advance with provincial Regulatory Authorities to determine whether their sludge quality is appropriate for land application.

12.9.2 Sanitary Landfill

Sanitary landfilling of sludge, either separately or along with municipal solid waste, may be an acceptable means of ultimate sludge disposal.

The sludge must be stabilized prior to landfilling and daily soil cover must be provided. In Nova Scotia, organics are not normally permitted to be landfilled, however, it could be used to help grow grass or cover material. Sludge dewatering may be required prior to landfilling.

12.9.3 Incineration

Sludge incineration can be achieved in a multiple-hearth furnace or fluidized bed incinerators. Particular attention is drawn to proper air pollution control of the stack gases to conform to the regulations of the Regulatory Authorities. Sludge dewatering is required prior to incineration.

12.9.4 Land Reclamation

Wastewater sludge can be used to reclaim strip-mine spoils or other low-quality land. Particular attention is drawn to the potential for water contamination and excessive accumulation of trace elements.

12.9.5 Energy/Resource Recovery

Energy and resource recovery processes include:

- Recovery and recycling of marketable constituents of sludge or sludge incinerator ash.
- Co-incineration of sludge with combustible solid waste to generate power or steam.
- Pyrolysis of sludge to produce useful by-products such as fuel gases, oils, tars, or activated charcoal.

If such techniques are used, a detailed description of the process and design data should accompany the plans.

12.10 References

Alberta Environment. *Guidelines for Municipal Wastewater Irrigation*. Edmonton, AB: Municipal Program Development Branch Environmental Sciences Division Environmental Service, 2000. <https://open.alberta.ca/dataset/86025ac1-8a7c-426a-b0d6-a86cc4502520/resource/378e6323-3ec6-43e7-b238-d2e19230fc07/download/2425423-2000-guidelines-for-wastewater-irrigation.pdf>.

Canadian Council of Ministers of the Environment. *Guidelines for Compost Quality. Canadian Council of Ministers of the Environment*, 2005. <https://ccme.ca/en/res/compostgdlns_1340_e.pdf>.

Canadian Food Inspection Agency. *Fertilizers Act*. Government of Canada, 2020. <https://laws-lois.justice.gc.ca/eng/acts/f-10/>.

Canadian Standards Association. *CAN/CGA-B105-M93 Code for Digester Gas and Landfill Gas Installations*. Toronto, CSA Group. 2007

Environment Canada. *Manual for Land Application of Treated Municipal Wastewater and Sludge*, EPS 6-EP-84-1. Environment Canada, 1984.

NFPA. *820 Standard for Fire Protection in Wastewater Treatment and Collection Facilities*. NFPA, 2020.4

Nova Scotia Environment and Labour. *Guidelines for Land Application and Storage of Biosolids in Nova Scotia*. NSE, 2004.

Ontario Ministry of the Environment. *Design Guidelines for Sewage Works*. Government of Ontario, 2019. Retrieved from <https://www.ontario.ca/document/design-guidelines-sewage-works-0>.

Tchobanoglous, George, Burton, Franklin L., Stensel, H. David, and Metcalf & Eddy. *Wastewater Engineering: Treatment and Reuse*. 4th Ed. / Revised by George Tchobanoglous, Franklin L. Burton, H. David Stensel. ed. McGraw-Hill Series in Civil and Environmental Engineering. Boston; Montreal: McGraw-Hill, 2003.

U.S. Environmental Protection Agency, *Handbook for Stream Sampling for Waste Load Allocation Application*, EPA/G25/6-86/013, 1986. https://nepis.epa.gov/Exe/ZyPDF.cgi/30004BG9.PDF?Dockey=30004BG9.PDF

U.S. Environmental Protection Agency. *Biosolids Applied to Land: Advancing Standards and Practices*. Washington, DC: USEPA, 2002.

U.S. Environmental Protection Agency. *Process Design Manual for Sludge Treatment and Disposal*. Washington, DC: USEPA, 2002.

United States Environmental Protection Agency. *Part 503- Standards for the use or disposal of sewage sludge. Clean Water Act*, February 1993. <https://www.ecfr.gov/current/title-40/chapter-I/subchapter-O/part-503>

Wastewater Committee of the Great Lakes - Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers. *Recommended Standards for Wastewater Facilities*. Health Research, Inc., Health Education Services Division, 2014. <https://www.health.state.mn.us/communities/environment/water/docs/tenstates/tenstatestan2014.pdf>.

Water Pollution Control Federation. *Sludge Dewatering, Manual of Practice No.20*. Washington, DC: USEPA, 1983.

Appendix A

Additional Considerations for Pre-design & Final Design Reports

Table of Contents

Appendix A

Additional Considerations for Pre-design & Final Design Reports

Table 1.1 in these Guidelines provides a checklist of wastewater system components that typically should be included in a Pre-design Report. The requirements can be determined by the System Owner by indicating which components are applicable to the project and serve to determine the contents for a specific project to be submitted to the Regulatory Authority for review.

The following provides a list of specific tasks that may be required under each of the selected wastewater collection and wastewater treatment components.

A.1 Pre-design Report

Introduction:

- Purpose.
- Reasons for report and circumstances leading up to report.
- Scope.
- Coordination of recommended project with approved comprehensive master plan and guideline for developing the report.

Climate Change Risk Assessment:

- Screening-level assessment and/or risk assessment (if applicable) addressing the impacts of climate change on the project. Refer to Chapter 2 and Appendix F for further guidance.

Background:

Present only appropriate history.

- General:
  - Existing area, expansion, annexation, inter-municipal service, and ultimate area.
  - Drainage basin and portion covered.
  - Population growth, trends, increase during design life of facility (graph), etc.

- Residential, commercial, and industrial land use, zoning, population densities, industrial types, and concentrations.
- Topography, general geology, and effect on project.
- Effect on project of meteorology, precipitation, runoff, flooding, etc.
- Total period for which project is to be studied.

- Economics:
  - Assessed valuation, tax structure, tax rates, and portions for residential, commercial, and industrial property.
  - Employment from within and outside service area.
  - Transportation systems and effect on commuter influx.
  - Exempt properties such as churches, and agricultural exhibition.
  - Costs of present water and wastewater services.

- Regulations:
  - Existing ordinances, rules, and regulations including defects and deficiencies, etc.
  - Recommended amendments, revisions or cancellation and replacement.
  - Sewer-use ordinance (toxic, aggressive, volatile, substances, etc.).
  - Surcharge based on volumes and concentration for industrial wastewaters.
  - Existing contracts and agreements (e.g., inter-municipal).
  - Enforcement provisions including inspection, sampling detection, penalties, etc.

- Hydraulic Capacity:
  - The following flows for the design year shall be identified and used as a basis for design for sewers, lift stations, WWTPs, treatment units, and other wastewater handling plants. Where any of the terms defined in this Section are used in these design standards, the definition contained in this Section applies.

- Design Average Flow:
  - The design average flow is the average of the daily volumes to be received for the continuous 12-month period expressed as a volume per unit time. That said, the design average flow for plants having critical seasonal high hydraulic loading periods (e.g., recreational areas, campuses, and industrial plants) shall be based on the daily average flow during the seasonal period.

- Design Maximum Day Flow:
  - The design maximum day flow is the largest volume of flow to be received during a continuous 24-hour period expressed as a volume per unit time.

- Design Peak Hourly Flow:
  - The design peak hourly flow is the largest volume of flow to be received during a 1-hour period expressed as a volume per unit time.

- Design Peak Instantaneous Flow:
  - The design peak instantaneous flow is the instantaneous maximum flowrate to be received.

- Design Minimum Day Flow:
  - The design minimum day flow is the smallest volume of flow to be received during a 24-hour period during dry weather when I&I are at a minimum, expressed as a volume per unit time.

Existing Plant Evaluation:

- Existing Collection:
  - Inventory of existing sewers.
  - Isolation from water supply wells.
  - Adequacy to meet project needs (structural condition, hydraulic capacity tabulation).
  - Gauging and infiltration tests (tabulate).
  - Overflows and required maintenance, repairs, and improvements.
  - Outline repair, replacement, and storm water separation requirements.
  - Evaluation of costs for treating I&I vs. costs for rehabilitation of system.
  - Establish renovation priorities (if selected).
  - Present recommended annual program to renovate sewers.
  - Indicate required annual expenditure.

- Existing Treatment Plant:
  - Area for expansion.
  - Surface condition.
  - Subsurface conditions.
  - Isolation from habitation.
  - Isolation from water supply structures.
  - Enclosure of units, winter conditions, odour control, landscaping, etc.
  - Flooding (predict elevation of 25- and 100-year flood stage including climate change considerations).

- Existing Process Facilities:
  - Capacities and adequacy of units (tabulate).
  - Relationship and/or applicability to proposed project.
  - Age and condition.
  - Adaptability to different usages.
  - Structures to be retained, modified, or demolished.
  - Outfall.

- Existing Wastewater Characteristics:
  - Water consumption (from records) total, unit, and industrial.
  - Wastewater flow pattern, peaks, and total design flow.
  - Physical, chemical, and biological characteristics and concentrations.
  - Residential, commercial, industrial, infiltration fractions, considering organic solids, toxic aggressive, etc., tabulate each fraction separately and summarize.

Proposed Project:

- Collection System:
  - Inventory of proposed additions.
  - Isolation from water supply well, reservoirs, facilities, etc.

- Area of services.
- Unusual construction problems.
- Utility interruption and traffic interference.
- Restoration of pavements, lawns, etc.
- Basement flooding prevention during power outage.

- Site Requirements:
  - Comparative advantages and disadvantages as to cost, hydraulic requirements, flood control, accessibility, enclosure of units, odour control, landscaping, etc., and isolation with respect to potential nuisances and protection of water supply facilities.

- Wastewater Characteristics:
  - Character of wastewater necessary to ensure amenability to process selected.
  - Need to pretreat industrial wastewater before discharge to sewers.
  - Portion of residential, commercial, industrial wastewater fractions to comprise projected growth.

- Receiving Water Considerations and Assimilative Capacity:
  - Wastewater discharges upstream.
  - Receiving water base flow (utilize critical flow as specified by Regulatory Authority).
  - Characteristics (concentrations) of receiving waters.
  - Downstream water uses including water supply, recreation, agricultural, industrial, etc.
  - Impact of proposed discharge on receiving waters (near and long-term discharge).
  - Tabulate assimilative capacity requirements.
  - Listing of effluent characteristics.
  - Tabulation and correlation of plant performance vs. receiving water requirements.

Alternatives:

Alternatives should consider such items as regional solution, optimum operation of existing plants, flow and waste reduction, location of plants, phased construction, necessary flexibility and reliability, sludge disposal, alternative treatment sites, alternative processes, and institutional arrangements.

- Alternate Process and Site:
  - Describe and delineate (line diagrams).
  - Preliminary design for cost estimates.
  - Estimates of project cost (total) dated, keyed to construction cost index, escalated, etc.
  - Advantages and disadvantages of each.
  - Individual differences, requirements, and limitations.
  - Characteristics of process output.
  - Comparison of process performances.
  - Operation and maintenance expenses.
  - Annual expense requirements (tabulation of annual operation, maintenance, personnel, and debt obligation for each alternate).
  - Environmental assessment of each.

- Selected Process and Site:
  - Identify and justify process and site selected.
  - Adaptability to future needs, conditions, and climate.
  - Environmental assessment.
  - Outfall location.
  - Describe immediate and deferred construction.

Project Financing:
- Review applicable financing methods.
- Effect of provincial and federal funding.
- Assessment by frontage, area unit or other benefit.
- Charges by connection, occupancy, readiness-to-serve, water consumption, industrial wastewater discharge, etc.
- Existing debt service requirements.
- Annual financing and bond retirement schedule.
- Tabulate annual operating expenses.
- Show anticipated typical annual charge to user and non-user.
- Show how representative properties and users are to be affected.

Legal and Other Considerations:
- Needed enabling legislation, ordinances, rules, and regulations.
- Contractual considerations for inter-municipal cooperation.
- Public information and education.
- Statutory requirements and limitations.

Appendices, Technical Information, and Design Criteria:
- Collection System:
  - Design tabulations (flow, size, velocities, pipe materials, etc.).
  - Regulatory Authority or overflow design.
  - Pump station calculations (including energy requirements).
  - Special appurtenances.
  - Stream crossings.
  - System map (report size).

- Process Facilities:
  - Criteria selection and basis.
  - Hydraulic and organic loadings (minimum, average, maximum, and effect).
  - Unit dimensions.
  - Rates and velocities.
  - Detentions.
  - Concentrations.
  - Recycle.
  - Chemical additive control.
  - Physical control.

- Removals, effluent concentrations, etc. Include a separate tabulation for each unit to handle solid and liquid fractions.
- Waste sludge processing and disposal including quantity estimates.
- Process (or Piping) and Instrumentation Diagram (P&ID).
- Energy requirement.
- Flexibility.

- Process Diagrams:
  - Process configuration, interconnecting piping, processing, flexibility, etc.
  - Hydraulic profile.
  - Organic loading profile.
  - Solids control system.
  - Solids profile.
  - Flow diagram with capacities, etc.

- Space for Personnel, Laboratories, and Records:
  - Provide necessary space for required personnel and laboratory facilities.
  - Provide readily available space for record keeping.

- Chemical Control:
  - Processes needing chemical addition.
  - Chemicals and feed equipment.
  - Chemical storage and containment.
  - Tabulation of amounts and unit and total costs.

- Support Data:
  - Outline unusual specifications, construction materials and construction methods.
  - Maps, photographs, and diagrams (report size).
  - Other.

Supplemental Information:
- Treated Effluent to Land (where applicable).
- In addition to the required Pre-design Report, when treated effluent is proposed to be discharge on land, the designer shall include supplemental information, as outlined below. This information shall include any material that is pertinent about the location, geology, topography, hydrology, soils, areas for future expansion, and adjacent land use.

- Location:
  - The following supplement information is required to be submitted with the Pre-design Report:
    - A copy of the topographic map of the area showing the exact boundaries of the proposed application area.
    - A topographic map of the total area owned by the applicant at a scale of approximately 1-in-10,000. It should show all buildings, the waste disposal system, the spray field boundaries, and the buffer zone. An additional map should show the spray field topography in detail with a contour interval of 0.5 m and include buildings and land use on adjacent lands within 400 m of the project boundary.

    - All water supply wells which might be affected shall be located and identified as to use (e.g., potable, industrial, agricultural, and ownership (public, private, etc.)).
    - All abandoned wells, shafts, etc., shall be located and identified. Pertinent information therein shall be furnished.
    - Separation distances shall comply with requirements of Sections 3.12 and 3.13.

- Geology:
  - The geologic formations (name) and the rock types at the site.
  - The degree of weathering of the bedrock.
  - The local bedrock structure including the presence of faults, fractures, and joints.
  - The character and thickness of the surficial deposits (residual soils and glacial deposit).
  - In limestone terrain, additional information about solution openings and sinkholes is required.
  - The source of the above information must be indicated.

- Hydrology:
  - The depth to seasonal high-water table (perched and/or regional) must be given, including an indication of seasonal variations. Static water levels must be determined at each depth for each aquifer in the depth under concern. Critical slope evaluation must be given to any differences in such levels.
  - The direction of groundwater movement and the point(s) of discharge must be shown on one of the attached maps.
  - Chemical analyses indicating the quality of groundwater at the site must be included.
  - The source of the above data must be indicated.
  - The following information shall be provided from existing wells and from such test wells as may be necessary:
    - Construction details, where available: depth, well log, pump capacity, static levels, pumping water levels, casing, grout material, and such other information as may be pertinent.
    - Groundwater quality (e.g., nitrates, TN, chlorides, sulphates, pH, alkalinities, total hardness, coliform bacteria, etc.).
    - A minimum of one groundwater monitoring well must be drilled for the protection of potable water wells or as determined by the Regulatory Authority having jurisdiction, in each dominant direction of groundwater movement and between the project site and public well(s) and/or high-capacity private wells, with provision for sampling at the surface of the water table and at 1.5 m below the water table at each monitoring site. The location and construction of the monitoring well(s) must be approved by the Regulatory Authority. These may include one or more of the test wells where appropriate.

- Soils:
  - A soils map of the spray field should be furnished, indicating the various soil types. This may be included on the large-scale topographic map. Soils information can normally be secured through the Federal Department of Energy Mines and Resources, the Federal Department of Agriculture, or the applicable provincial department.
  - The soils should be named, and their texture described.
  - Slopes and agricultural practice on the spray field are closely related. Slopes on cultivated fields should be limited to 4%.
  - Slopes on sodded fields should be limited to 8%. Forested slopes should be limited to 8% for year-round operation, but some seasonal operation slopes up to 14% may be acceptable.

- The thickness of soils should be indicated. Method of determination should be included.
- Data should be furnished on the exchange capacity of the soils. In case of industrial wastes particularly, this information must be related to special characteristics of the wastes.
- Information must be furnished on the internal and surface-drainage characteristics of the soil materials. This includes the soil's infiltration capacity and permeability.
- Proposed application rates should take into consideration the drainage and permeability of the soils, the discharge capacity, and the distance to the water table.

- Agricultural Practice:
  - The present and intended soil-crop management practices, including forestation, shall be stated.
  - Pertinent information shall be furnished on existing drainage systems.
  - When cultivated crops are anticipated, the kinds used, and the harvesting frequency should be given; the ultimate use of the crop should also be given. See Section 11.3.3.4 for crop considerations.

- Adjacent Land Use:
  - Present and anticipated use of the adjoining lands, up to 400 m from the site, must be indicated. This information can be provided on one of the maps and may be supplemented with notes.
  - The plan shall show existing and proposed screens, barriers, or buffer zones to prevent blowing spray from entering adjacent land areas.
  - If expansion of the facility is anticipated, the lands which are likely to be used for expanded spray fields must be shown on the map.

• 2 Design Report

Table 1.2 in these Guidelines provides a checklist of components that typically should be included in a Design Report. The requirements can be determined by the System Owner by indicating which components are applicable to the project and serve to determine the contents for a specific project to be submitted to the Regulatory Authority for review and approval. The following provides a list of specific tasks that may be required under each of the selected wastewater collection and wastewater treatment components.

Climate Change:

- A risk assessment, including all pertinent information (e.g., climate projections, data sources, evaluation methodology, results, identification/prioritization of adaptations strategies, etc.), shall be included in the Detailed Design Report. The climate resiliency strategies incorporated into design should be clearly stated in the Detailed Design Report. For reporting purposes, only the risk assessment may be included in the Detailed Design Report if both a screening-level assessment and risk assessment were undertaken. If only a screening-level assessment was undertaken, then it should be included in the Detailed Design Report. Clear reasoning should be provided in the case that the conclusions of the screening level assessment negated the completion of a risk assessment and climate resilient strategies were not incorporated into design (this may be presented the screening-level assessment). Refer to Chapter 2 and Appendix F for further guidance.

Collection System:

- Detailed design tabulations (flow, size, velocities, etc.).
- Regulatory Authority or overflow design calculations.
- Detailed pump station calculations (including energy requirements).
- Special appurtenances.

- Stream crossings.
- System map (report size).

Process Facilities:
- Hydraulic and organic loadings (minimum, average, maximum, and effect).
- Detailed calculations used to determine:
  - Unit dimensions.
  - Rates and velocities.
  - Detentions.
  - Concentrations.
  - Recycle.
  - Removals, effluent concentrations, etc. Include a separate tabulation for each unit to handle solid and liquid fractions.
  - Energy requirement.
  - Flexibility.
- Chemical requirements and control.

Process Diagrams:
- Process configuration, interconnecting piping, processing, flexibility, etc.
- Hydraulic profile.
- Organic loading profile.
- Solids control system.
- Solids profile.
- Flow diagram with capacities, etc.

Laboratory:
- Physical and chemical tests and frequency to control process.
- Time for testing.
- Space and equipment requirements.
- Personnel requirements (number, type, qualifications, etc.).

Operation and Maintenance:
- Routine and special maintenance duties.
- Time requirements.
- Tools, equipment, vehicles, safety, etc.
- Personnel requirements (number, type, qualifications, salaries, benefits (tabulate)).
- Maintenance workspace and storage.

Office Space for Administrative Personnel and Records:
- Provide necessary space for required personnel and readily available space for record keeping.

Personnel Service – Locker Room and Lunchroom:
- Provide necessary locker room and lunchroom space.

Chemical Control:

- Process needing chemical addition.
- Chemicals and feed equipment.
- Tabulation of amounts and unit and total costs.

Collection System Control:

- Cleaning and maintenance.
- Regulatory Authority and overflow inspection and repair.
- Flow gauging.
- Industrial sampling and surveillance.
- Regulation enforcement.
- Equipment requirements.
- Trouble-call investigation.
- Personnel requirements (number, type, qualifications, etc.).

Control Summary:

- Personnel.
- Equipment.
- Chemicals.
- Utilities (list power requirements of major units).
- Programmable logic controller and Supervisory Control and Data Acquisition (SCADA) details.
- Summation.

Support Data:

- Outline unusual specifications, construction materials, and construction methods.
- Maps, photographs, diagrams (report size).
- Other.

Appendices:

- Related data not necessary to an immediate understanding of the Design Report should be placed in the appendices.

Appendix B

Operation & Maintenance Workforce Requirements

Table of Contents

List of Figures

Appendix B

Operation & Maintenance Workforce Requirements

B.1 Operation & Maintenance Workforce

Figure B.1 outlines overall workforce requirements for each class of WWTP over a wide range of average design flows. The information presented is to assist those seeking to project future WWTP staffing requirements as a basis for planning of operations training programs. The data can also be used as a guide for staffing requirements for individual conventional WWTPs, provided recognition is given to the "average" nature of the estimating data, and judgement is applied regarding specific local circumstances.

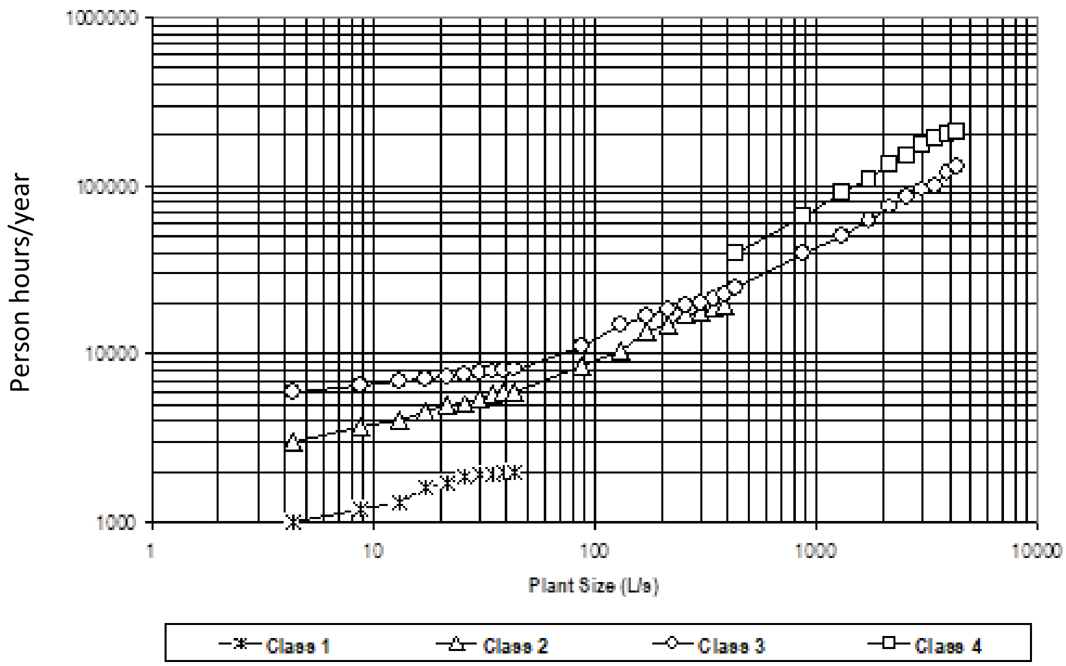


Figure B.1: Workforce Requirements vs. Plant Size

B.1.1 Wastewater Treatment Operator Core Competencies

The following Section lists the core competencies for WWTP operators. The following list is not broken down for each class level of operator, therefore, there may be job tasks and capabilities that are common in one level and not in others. Considering operators may move from one facility to another, they should have a basic understanding of the common job tasks performed at various facilities throughout the United States and Canada. The Association of Boards of Certification (ABC) skill requirements for collection and treating system personnel have been included with permission from the ABC.

The core competencies are clustered into eight job duties that are performed by each level of WWTP operator:

- Establish safety plans and apply safety procedures.
- Monitor, evaluate, and adjust treatment processes.
- Evaluate physical characteristics of waste stream.
- Perform and interpret laboratory analyses.
- Operate equipment and evaluate operation of equipment.
- Perform preventive and corrective maintenance.
- Perform administrative duties.

Following each job duty is a listing of the job tasks and capabilities, in alphabetical order, that are associated with that duty.

B.1.1.1 Establish Safety Plans & Apply Safety Procedures

Plans and Procedures Include:

- Blood borne pathogens.
- Chemical hazard communication.
- Confined space entry.
- Electrical grounding.
- Facility upset.
- First-aid.
- General safety and health.
- Lifting.
- Lock-out/tag-out.
- Personal hygiene.
- Personal protective equipment.
- Respiratory protection.
- Slips, trips, and falls.
- Spill response.
- Traffic control.
- Transportation.

Required Capabilities:

- Ability to assess likelihood of disaster occurring.
- Ability to communicate safety hazards verbally and in writing.
- Ability to demonstrate safe work habits.
- Ability to follow written procedures.

- Ability to identify potential safety hazards.
- Ability to recognize unsafe work conditions
- Ability to select and operate safety equipment.
- Knowledge of emergency plans.
- Knowledge of potential causes and impact of disasters on facility.
- Knowledge of safety regulations.

B.1.1.2 Monitor, Evaluate, & Adjust Processes

Processes Include:

- Activated sludge.
- Chemical addition.
- Clarifiers.
- Disinfection.
- Grit removal.
- Pumping of main flow.
- Screens.
- Solids handling.

Required Capabilities:

- Ability to adjust chemical feed rates, flow patterns, and process units.
- Ability to calculate dosage rates.
- Ability to confirm chemical strength.
- Ability to evaluate, diagnose, and troubleshoot process units.
- Ability to interpret SDS.
- Ability to maintain processes in normal operating conditions.
- Ability to measure and prepare chemicals.
- Ability to perform basic math and process control calculations.

Required Knowledge:

- Knowledge of biological science.
- Knowledge of general chemistry.
- Knowledge of general electrical principles.
- Knowledge of mechanical principles.
- Knowledge of normal chemical range.
- Knowledge of personal protective equipment.
- Knowledge of principles of measurement.
- Knowledge of proper application, handling, and storage of chemicals.
- Knowledge of proper lifting procedures.
- Knowledge of regulations.
- Knowledge of wastewater treatment concepts and treatment processes.

B.1.1.3 Evaluate Physical Characteristics of Waste Stream

Characteristics Include:

- Colour.
- Flow pattern.

- Foam.
- Mixing pattern.
- Odour.
- Solids concentration.
- Volume.

Required Capabilities:

- Ability to communicate observations verbally and in writing.
- Ability to discriminate between normal and abnormal conditions.
- Knowledge of normal characteristics of wastewater.

B.1.1.4 Perform & Interpret Laboratory Analyses

Analyses Include:

- 5-day biochemical oxygen demand.
- Ammonia.
- Chlorine residual.
- Coliform.
- Dissolved oxygen.
- pH.
- Settleable solids.
- Temperature.
- Total suspended solids.
- Volatile suspended solids.

Required Capabilities:

- Ability to calibrate instruments
- Ability to follow written procedures.
- Ability to interpret SDS.
- Ability to perform laboratory calculations.
- Ability to recognize abnormal analytical results.

Knowledge of:

- Knowledge of biological science.
- Knowledge of general chemistry.
- Knowledge of laboratory equipment and procedures.
- Knowledge of normal characteristics of wastewater.
- Knowledge of principles of measurement.
- Knowledge of proper chemical handling and storage.
- Knowledge of quality control and assurance practices.
- Knowledge of safety regulations.
- Knowledge of sampling procedures.
- Knowledge of *Standard Methods for the Examination of Water and Wastewater* from the American Public Health Association, the AWWA, and the WEF.

B.1.1.5 Operate Equipment

Equipment Includes:

- Backflow prevention devices.
- Blowers and compressors.
- Chemical feeders.
- Computers.
- Digesters.
- Drives.
- Electronic testing equipment.
- Engines.
- Generators.
- Heavy vehicles.
- Hydraulic equipment.
- Instrumentation.
- Motors.
- Pneumatic equipment.
- Pumps.
- Valves.

Required Capabilities:

- Ability to adjust, evaluate, and monitor operation of equipment.
- Knowledge of electrical and mechanical principles.

Required Knowledge:

- Knowledge of function of tools.
- Knowledge of safety regulations.
- Knowledge of start-up and shut-down procedures.
- Knowledge of wastewater treatment concepts.

B.1.1.6 Evaluate Operation of Equipment

Characteristics Include:

- Read meters.
- Read charts.
- Read pressure gauges.
- Check speed of equipment.
- Measure temperature of equipment.
- Inspect equipment for abnormal conditions.

Required Capabilities:

- Ability to discriminate between normal and abnormal conditions.
- Ability to monitor and adjust equipment.
- Ability to report findings.

Knowledge of:
- Knowledge of electrical and mechanical principles.
- Knowledge of process control instrumentation.

B.1.1.7 Perform Preventative & Corrective Maintenance
Equipment Includes:
- Blowers and compressors.
- Chemical feeders.
- Generators.
- Instrumentation.
- Motors.
- Pumps.

Required Capabilities:
- Ability to assign work to proper trade.
- Ability to calibrate equipment.
- Ability to diagnose and troubleshoot units.
- Ability to differentiate between preventive and corrective maintenance.
- Ability to discriminate between normal and abnormal conditions.
- Ability to follow written procedures.
- Ability to perform general maintenance.
- Ability to record information.

Knowledge of:
- Knowledge of electrical and mechanical principles.
- Knowledge of facility O&M.
- Knowledge of safety regulations.
- Knowledge of start-up and shut-down procedures.

B.1.1.8 Perform Administrative Duties
Tasks Include:
- Control employee work activities.
- Establish recordkeeping systems for facility.
- Plan and organize work activities.
- Record information relating to facility performance.
- Respond to complaints.
- Write internal, provincial, and federal reports.

Required Capabilities:
- Ability to determine what information needs to be recorded.
- Ability to evaluate employee and facility performance.
- Ability to interpret and transcribe data.
- Ability to organize information and follow written procedures.
- Ability to perform basic math.
- Ability to translate technical language into common terminology.

Knowledge of:

- Knowledge of facility operation and maintenance.
- Knowledge of monitoring and reporting requirements.
- Knowledge of principles of general communication, management, public relations, and supervision.
- Knowledge of recordkeeping functions and policies.
- Knowledge of regulations.

B.1.2 Job Descriptions

Job descriptions for the types of personnel commonly employed for the O&M of conventional wastewater treatment systems are defined in the manual *Estimating Costs and Manpower Requirements for Conventional Wastewater Treatment Facilities* from the USEPA (Contract No. 14-12-462).

A job description for a specific occupation may include details from several of the core competency job duties from Section B.1.1 depending upon the flexibility required, however, a good job description should include but is not necessarily limited to the following:

- List items or processes that an individual must operate.
- State if monitoring of gauges or meters is required.
- Discuss interpreting of any meter or gauge readings for process control actions.
- List any logs or records to be maintained.
- Outline any maintenance duties required.
- State any other title that an individual might carry.
- Discuss decision making requirements.
- State responsibilities and authority given to an individual in the job being described.
- List any report or budget functions that must be performed.
- Discuss any supervisory or inspection functions.

B.1.3 Suggested References

The following manuals are recommended for operators looking for references on how to operate WWTPs.

California State University, Sacramento:

- *Operation of Wastewater Treatment Plants* (Volumes 1 and 2) from the California State University, Sacramento.
- *Advanced Waste Treatment* from the California State University, Sacramento.
- *Utility Management* from the California State University, Sacramento.

To order the latest edition, contact:
California State University
Office of Water Programs
Sacramento, 6000 J Street
Sacramento, CA 95819-6025
Phone: (916) 278-6142
Fax: (916) 278-5959
E-mail: wateroffice@csus.edu

Water Environment Federation:

- *Operation of Municipal Wastewater Treatment Plants: Manual of Practice No. 11* from the Water Environment Federation.
- *Design of Municipal Wastewater Treatment Plants: Manual of Practice No. 8* from the Water Environment Federation.
- *Certification Study Guide for Wastewater Treatment Personnel* from the Association of Boards of Certification and the Water Environment Federation.

For more in-depth references on specific aspects of wastewater treatment, contact the Water Environment Federation for a complete list of Manuals of Practice.

To order the latest edition, contact:
Water Environment Federation
601 Wythe Street
Alexandria, VA 22314-1994
Phone: (800) 666-0206
Fax: (703) 684-2492
E-mail: pubs@wef.org

B.2 References

Alberta Environment. *Water and Wastewater Operators' Certification Guidelines*, March 2020. <https://open.alberta.ca/dataset/1a0a3944-e7a0-4fc4-b617-5be794531110/resource/7c741be1-c10a-42a1-8137-e753ea3ace86/download/aep-operators-certification-guidelines-2020-03.pdf>.

Sacramento State. *Advanced Waste Treatment*. 2006. <https://www.owp.csus.edu/courses/wastewater/advanced-waste-treatment.php>.

Sacramento State. *Operation of Wastewater Treatment Plants,* Volume II. 2007. <https://www.owp.csus.edu/courses/wastewater/operation-of-wastewater-treatment-plants-vol-ii.php>.

Sacramento State. *Operation of Wastewater Treatment Plants*, Volume I. <https://www.owp.csus.edu/courses/wastewater/operation-of-wastewater-treatment-plants-vol-1.php>.

Sacramento State. *Utility Management*. 2021. <https://www.owp.csus.edu/courses/management/utility-management.php>.

Water Environment Federation. *Manual of Practice 11 – Operation of Municipal Wastewater Treatment Plants*, Volumes I, II and III, 2007. <https://www.wefnet.org/ewef/images/MOP11/Frontmatter.pdf>.

Water Environment Federation. *Manual of Practice 8 – Design of Municipal Wastewater Treatment Plants*, Volumes I, II and III, 2010. <https://www.wefnet.org/ewef/images/mop8/mop8_frontmatter.pdf>.

Water Environment Federation. *Standard Methods for the Examination of Water and Wastewater*, 23rd Edition. American Public Health Association, American Water Works Association, and Water Environment Federation, 2017. <https://connect.wef.org/s/store#/store/browse/detail/a2n1S000000KVk1>.

Appendix C

Treatment Process Control

Table of Contents

List of Figures

Appendix C

Treatment Process Control

C.1 Instrumentation & Controls

The requirements for treatment process instrumentation and controls will depend on the WWTP size, location, and process type. In general, instrumentation and controls should provide safe and efficient manual and automatic operation of all parts of the plant, with minimal operator effort, and all automatic controls should be provided with manual back-up systems.

When making decisions relating to instrumentation and control, the following factors should be considered:

- Plant size and complexity.
- Regulatory Authority requirements.
- Hours of attended operation.
- Parameters which are useful for process control.
- Primary element reliability.
- Primary element location.
- Whether each type of equipment should be manually, remotely, and/or locally controlled.
- Data storage and recording requirements.
- Whether data acquisition should be central or distributed.
- Potential chemical and energy savings.
- Safety.

C.1.1 Process Control Techniques

There are two main types of process control techniques within a WWTP. These include manual control and online control. Under the manual control system there is limited automatic control and the operator is responsible for decisions and actions. Online control involves a multi-purpose computerized system with limited scope for modification or a dedicated purpose system with standard hardware and customized software.

Whether process control involves manual or online control, or a combination of both, the operation and maintenance manual shall fully describe specific process control techniques.

C.1.2 Remote Control vs. Local Control

Where some parts of a plant may be operated or controlled from a remote location. Local control stations should be provided and shall include the provision for preventing operation of the equipment from the remote location. The designer must decide which equipment will be controlled locally and which will be controlled from a remote location, and whether control will be automatic or manual.

C.1.3 Supervisory Control & Data Acquisition

Supervisory control and data acquisition systems should be used to control and monitor wastewater treatment equipment and wastewater collection systems. If radio-based telemetry systems are used, special attention should be given to the design and layout to eliminate any potential for Radio Frequency Interference (RFI).

Ensure that support for any proprietary software will be available from the vendor once the project has been accepted.

C.1.4 Recommended Parameters

For effective operation of larger WWTPs, the following parameters should be measured, if present. Some may not be required or relevant for smaller facilities or may be different.

- Flowrate for raw wastewater, by-pass flows, train-specific flows, and final effluent flow.
- Dissolved oxygen levels.
- Return activated sludge and WAS flows.
- Sludge blanket levels and sludge concentrations.
- Chemical dosages.
- Ultraviolet transmittance.
- Effluent pH.
- Raw and digested sludge flow, digester supernatant flows.
- Digester gas production.
- Anaerobic digester temperature.
- Hazardous gas monitoring.

C.1.5 Types of Instruments

The different types of instruments that may be required to measure the previously mentioned parameters are classified as primary element devices, which alter a signal from a physical process to make it suitable for use by a transmitter. These devices are broken down into function groups with a brief description of the process application.

C.1.5.1 Flow Measurement

Magnetic Flowmeters (Mag Meters)

- Materials: the liner for the meter varies depending on the application. To resist moderate amounts of abrasion, use polyurethane, butyl rubber, neoprene, or polytetrafluoroethylene. Where corrosion is likely to occur, use ceramic or polytetrafluoroethylene. Stainless steel electrode material should be used where corrosion risk is high. Hastelloy electrode material should be used where corrosion risk is high.
- Installation: generally requires five straight pipe diameters upstream of the meter and three downstream of

the meter, free of valves or fittings. Meters may be installed on horizontal, vertical, or sloping lines. Electrodes must be in the horizontal plane for continuous contact with the fluid or slurry being metered. The operating velocity required is 1.0 to 10 m/s for liquids without solids and 1.5 to 7.5 m/s for liquids containing solids. When used to meter liquids containing solids, a continuous electrode cleaner or clean out tee should be installed.
- Applications: these meters are suitable for influent wastewater, primary sludge, RAS, WAS, digested sludge and final effluent. These meters should not be used for digester gas or liquid streams with a solids content greater than 10% by weight.

Ultrasonic Flowmeters
- Flowmeter Construction: the flowmeter consists of an electronics housing, transducers, and pipe section. These can also be fitted to existing pipes by drilling holes for the transducer hardware or by using external transducers on the outside of the pipe. When installed on existing pipes, the pipe material should be checked to assure it will not dampen the sonic signal as this will adversely affect performance.
- Installation: generally requires 10 to 20 straight pipe diameters upstream of the meter and five downstream of the meter, free of valves or fittings. Meters can be installed on horizontal, vertical, or sloping lines as long as the pipe sections are always full. The operating velocity required for these meters will be between 1.0 to 10 m/s.
- Applications: transmittance styles are not recommended for influent wastewater, primary sludge, thickened sludge, nitrification RAS, or nitrification WAS. Reflective styles are not recommended for primary effluent, secondary clarifier effluent final effluent or process wash water.

Turbine Flowmeters
- Flowmeter Construction: the flowmeter usually consists of meter body with rotor blades and a magnetic pickup. The pickup is often connected to electronic display units or a totalizer.
- Installation: installation of turbine flow meters generally require a minimum of stream of the meter free of valves or fittings. Meters may be installed on horizontal or vertical pipelines.
- Applications: turbine flow meters are recommended for applications involving natural gas, compressed digester gas.

Flumes and Weirs (Parshall Flume)
- Installation: affected by upstream channel arrangement; at least 10 channel widths upstream are recommended. Must be installed level.
- Applications: open channel flow measurement.

C.1.5.2 Suspended Solids Measurement (Turbidity)
- Installation: installation details for turbidity analyzers are unique to each manufacturer. The manufacturer's recommendations should be followed.
- Applications: turbidity analyzers are recommended for applications involving SS concentrations less than 100 mg/L.

C.1.5.3 Suspended Solids Measurement (Optical)
- Installation: unique to each manufacturer. The manufacturer's recommendations should be followed.
- Applications: recommended for solids concentrations from 20 mg/L to 8% (e.g., RAS, WAS, and mixed liquor).

C.1.5.4 Dissolved Oxygen Measurement (Galvanic)

- Installation: installation details for DO analyzers are usually related to the choice of placement of the analyzer in the process fluid. The analyzers generally require frequent maintenance, and this should be considered in determining the location for installation.
- Applications: recommended for oxygen concentrations from 0 to 20 mg/L.

C.1.5.5 Level Measurement

Ultrasonic Sensor

- Installation: the mounting location of the sensor is determined from restrictions established by the manufacturer. Typically, the sensor must be mounted a minimum distance above the high liquid level and should be located away from tank walls or other obstructions that may cause false echoes.
- Applications: used in many level and flow applications; not suitable for dense and persistent foam.

Float

- Installation: normally located in a stilling well when turbulence is expected.
- Applications: commonly used for high- and low-level alarms and for controlling pump starts and stops.

Capacitance

- Installation: the installation practices can vary, and the manufactures recommended installation should be used.
- Applications: for continuous level measurement and also as switches for alarms or start/stop control.

C.1.5.6 Pressure Measurement

Bourdon Tubes

- Installation: the installation practice should include the use of block and bleed valves.
- Applications: for pressure indication only. Pressure range 0 to 35,000 kPa (0 to 5,075 psi).

Bellows

- Installation: the installation practice should include the use of block and bleed valves.
- Applications: for pressure indication only. Pressure range 0 to 2,000 kPa (0 to 290 psi).

Diaphragms

- Installation: the installation practice should include the use of block and bleed valves. Transmitters should be installed according to manufacturer's recommendations. Temperature extremes should be avoided, and location should be as close as possible to the process measure site.
- Applications: for pressure indication or transmitter output. Pressure range 0 to 3,500 kPa (0 to 508 psi).

C.1.5.7 Temperature Measurement

Thermocouples

- Installation: installation with a thermowell is recommended.
- Applications: wide range of suitability, but check range of unit.

Resistance Temperature Detector

- Installation: installation with a thermowell is recommended.
- Applications: suitable for temperature ranges of 0 to 300°C, but check range of unit.

Thermistor

- Installation: installation with a thermowell is recommended.
- Applications: suitable for temperature ranges of 0 to 300°C, but check range of unit.

Thermal Bulb

- Installation: no special installation requirements.
- Applications: suitable for temperature ranges of 0 to 500°C, but check range of unit.

C.1.6 Process Controls

C.1.6.1 Lift Stations

Lift stations require simple and dependable instrumentation and control systems. The parameters that should be monitored include:

- Level.
- Flow.
- Pressure.
- Temperatures.
- Hazardous gas levels.
- Status and alarm conditions.

The monitoring and control requirements will vary for each individual case based on the size, location, and economic considerations.

C.1.6.1.1 Level Control

Single-speed pumps are typically controlled on start and stop levels. The level in the wet well increases to the point where a duty (or lead) pump is required to start. A lag and then a follow pump may be started if the level continues to increase. Pumping continues until a pump stop level is reached at which time the duty pump stops, or a series of stop levels will be reached and the lag and follow pumps stop prior to the duty pump. The pump start/stop control can be performed using any one of several level elements.
Variable speed pumps are typically controlled to maintain a level set point in the wet well. This requires a feedback type of control in which the measured variable (level) is compared to a set point value and the final control element is modulated to maintain the set point value. This requires reliable analog level measurement to function properly.

Regardless of the type of level control used, the system should include a separate low-level lock-out and high-level alarm.

C.1.6.1.2 Flow Monitoring

The flow-metering element should be selected carefully to ensure that there are no obstructions where clogging may occur. Bypass and isolation of the flow-metering element should be provided for routine maintenance activities. The flow-metering device should be connected to the control system or to a recording and totalizing device, or both. This provides for a record of flows out of the lift station. It can also be used to help identify possible problems in the discharge piping or force main.

C.1.6.1.3 Pressure Monitoring

Monitoring of the system discharge pressure can be useful in identifying possible problems in the discharge piping or force main and in monitoring pump performance. The pressure-metering device should be connected to the control system or to a recording device, or both.

C.1.6.1.4 Pumps & Motors

The following parameters should be monitored:

- Pump bearing temperature.
- Pump bearing vibration.
- Pump speed for variable speed applications.
- Pump discharge pressure.
- Motor voltage and current.
- Motor hours of operation.
- Motor bearing temperature.
- Motor windings temperature.

C.1.6.1.5 Alarms

Lift stations alarms should be as outlined in Section 4.2.12.

C.1.6.2 Mechanical Bar Screens

Three (3) methods are used to control the operation of mechanical bar screens:

- Simple manual start/stop, which requires the presence of the operator at the screen to start and stop the screen.
- Automatic activation by differential level, which uses the differential level across the screen to provide the start condition. The screen should run at least one complete screen cycle before stopping. The screen can be called to stop when the differential level is returned to a nil value, the final stop should be controlled using a sensor to determine cycle completion (e.g., limit switch, proximity sensor, or timer). In addition, a timer should be provided to initiate a cleaning cycle at regular intervals regardless of actual head loss. There should be an alarm signal with a head loss set at a point higher than the automatic start of the mechanical bar screen.
- Automatic activation by timer with differential level as emergency start condition, which uses the differential level across the screen to provide secondary start condition. The screen should run at least one complete screen cycle before stopping. The stop signal should be controlled using a sensor to determine cycle completion (e.g., limit switch, proximity sensor, or timer). There should be an alarm signal with a head loss set at a point higher than the automatic start of the mechanical bar screen.

C.1.6.3 Primary Treatment

C.1.6.3.1 Raw Sludge Pumping

Raw sludge pumping controls should incorporate the following features:

- Automatic or manual selection of duty pump.
- Online sludge density metering for control and monitoring.
- Online sludge flow monitoring and totalization.
- Online adjustable sludge density control.
- Individually selectable hopper pumping controls where required.
- Manual override for automatic controls.
- Online sludge blanket level monitoring and alarming.
- Online sludge pump monitoring and control.
- Sludge density feedback control for variable speed pumping with manual override.
- Online sludge pump speed monitoring and control with manual override.
- Online monitoring and control of primary tank scraper mechanisms.

C.1.6.3.2 Scum Pumping

The scum pumping controls should incorporate the following features:

- Automatic or manual selection of duty pump.
- Manual override for automatic controls.
- Online sludge blanket level monitoring and alarming.
- Automatic controls consisting of high and low scum tank level for starting and stopping scum pumps.
- High scum tank level alarm.
- Online scum pump speed monitoring and control with manual override.
- Scum tank flushing system for scum tank cleaning.

C.1.6.4 Secondary Treatment Controls

C.1.6.4.1 Dissolved Oxygen Control

Automatic DO control systems should be used to control the rate of air supply to aeration tanks. The following methods may be used:

- **Closed loop control (feedback control):** closed loop control consists of online DO analyzers providing feedback control to an airflow control device. The DO reading is compared to the DO set point. The resultant error signal is used to increase or decrease the rate of air flow to the aeration tanks. Automatic DO control should always be equipped with manual override.
- **Feed forward control:** feed forward control consists of a fixed volume of air being delivered to the aeration tanks for a given wastewater flowrate. This system may use online DO analyzers, but these are for monitoring only and do not provide feedback to the air flow control elements. Process status and alarms should be provided for DO level, blower operating parameters, air flow control elements.

C.1.6.4.2 Return Activated Sludge Control

The RAS pumping controls should incorporate the following features:

- Automatic or manual selection of duty pump.
- Variable speed pumping.
- Return activated sludge flow monitoring.
- Feedback control to match pumping rates to flow set points.
- Individual control of sludge return rate from individual final clarifiers.

- Manual override for automatic controls.
- Online monitoring of return sludge flowrate, pump speed, and status.

C.1.6.4.3 Waste Activated Sludge Control

The WAS pumping controls should incorporate the following features:

- Automatic or manual selection of duty pumps.
- Variable speed pumping.
- Waste activated sludge flow monitoring.
- Feedback control to match pumping rates to flow set points.
- Manual override for automatic controls.
- Online monitoring of waste sludge flowrate, pump speed, and status.

C.1.6.5 Chemical Control System

The most basic chemical dosing consists of a feeder or chemical metering pump that will dose at a fixed ratio to the influent or effluent flow of the plant, with no analyzer or feedback control. More specific chemical dosing may also be based on parameters such as return sludge flowrate or have analyzers and possibly feedback control. Chemical dosing requirements will vary widely depending on performance requirements and the specific process used.

C.1.6.6 Ultraviolet Disinfection Control System

The disinfection of final plant effluent using UV light in smaller plants is usually not adjusted, but in larger plants it is controlled using a feed-forward control system. Lamps and or lamp channels are turned on based on measured plant effluent flow. Ultraviolet transmittance analyzers may be used for monitoring system performance and can also be used to adjust the numbers of lamps or channels needed. Alarms for low UV intensity should be provided.

C.1.7 Control & Monitoring Systems

Control and monitoring systems can be a conventional system with recorders, indicators, switches, push buttons, indicating lights, control panels, etc. or it can be a computerized control system that utilizes various configurations of hardware and software to provide the control required. Computerized systems can be separated into two groups:

- Programmable logic controller systems.
- Distributed Control Systems (DCS).

C.1.7.1 Conventional Relay Control Systems

The conventional system is a passive system with limited automatic control, where the operator is responsible for decisions and actions that control the process.

C.1.7.2 Programmable Logic Controller Control Systems

The PLC based system is a multipurpose system with extensive scope for modification. The plant status, alarms, motor starters, meters and analyzers are all wired into input/output cards located in what are called racks. The racks may be mounted separately or placed in specific plant areas to reduce wiring costs. The input/output racks are associated with controllers that are programmed to perform the required process control functions. Changes can generally be made relatively easily by modification of or addition to the PLC programs.

Plant personnel require process information in real-time or in near real-time. The PLC systems accomplish this by means of a HMI. The HMI may be dedicated hardware and software or may come in the form of personal computers utilizing HMI software and connected to the PLC communications system. These systems vary widely in their capabilities and performance. The selection of hardware and software should be done carefully to assure current performance and future supportability and expendability.

C.1.8 Controls System Design Documents

Complete design documents should be prepared to ensure that construction can be completed correctly and also to properly record the system for future reference. The following are required in the design documents:

- Design and construction standards, specifications, and installation details.
- Panel sizing and general arrangements.
- Control system functional requirements.
- Control component and instrument data sheets.
- Operator interface and control hardware and software specifications including input/output lists.
- Control system programming and packaged system configuration standards, structure, and scope.

C.1.9 Control System Documentation

The following documents should be provided following completion of the control system:

- Record drawings to show any changes to the design and including any drawings produced during construction.
- Annotated listings of control system programs and packaged system configuration.
- Manufacturer's literature for all control and instrumentation components.
- Final wiring diagrams complete with wire and terminal coding.
- Motor control schematics.
- Instrument loop diagrams.
- Panel wiring and layout details.
- Programmable logic controller or DCS wiring schematics.
- Instrument calibration sheets.
- Operating instructions.

C.1.10 Controls System Training

Adequate training shall be provided to the plant O&M staff so that the system can be operated to meet the design criteria.

C.2 Laboratory Control

C.2.1 Sampling

Sample data is required for a number of reasons, including the following:

- Demonstrate that the effluent quality is within acceptable guidelines.
- Effectively control unit operations.

- Distribute charges for treatment among the various municipal districts and industries involved.
- Allow design of future treatment facilities as the plant is expanded.
- Predict the effect of the effluent on the receiving waters.

C.2.1.1 Sampling Procedures

Appropriate sampling point locations must be established independently for each treatment plant.

Some widely applicable principles for sampling are listed below:

- Sample point locations should have completely mixed wastewater or sludge, as far possible.
- Provide proper sampling equipment and use all safety precautions.
- Any floating materials, growths, deposits, etc. (including particles greater than 0.6 cm in diameter should be excluded when sampling).
- Immerse sample bottles in ice water to inhibit bacterial action if samples are to be kept for 1 hour or more prior to testing, or if they are to be shipped.
- Carefully label all sample bottles.
- Consider the plant's daily flow variation and detention time through the units so that influent and effluent samples relate to the same waste.

C.2.1.2 Frequency of Sampling

Required frequency of sampling depends on the:

- Treatment plant size and loading.
- Variability of the waste stream.
- Severity of possible effects on receiving water and public health.
- Regulatory Authority sampling requirements.
- Number of staff available.
- Hours of supervision.

Routine sampling to monitor plant performance and effluent quality should be undertaken regularly. More intensive sampling and testing may be required in the event of an upset, to assess unit operation performance and the effect of corrective action.

Figure C.1 presents a sample format for a laboratory sampling program. Note that different plants could have very different operational sampling programs, and that the Regulatory Authorities sampling requirements also vary from plant to plant.

	Settleable Solids	SS	BOD	Chlorine Residual	Grease	TDS	Coliform Bacteria	VSS	DO	TS	Total Volatile Solids	pH
Raw wastewater	C, D	C, D	C, D			C, W		C, D		C, W	C, W	
Primary effluent	C, D	C, D	C, D					C, D		C, W	C, W	
Secondary effluent	C, D	C, D	C, D		C,W			C, D		C, W	C, W	
Chlorine cont. tank				G, E								
Mixed liquor		C, D										
Plant effluent			C, W	G, D2		C, W	G, W		G, D			
Raw sludge										C, W	C, W	C, W
Digested sludge										G, W	G, W	G, W

Type of Sample
C: Composite sample
G: Grab sample

Frequency
D: Daily
W: Weekly
D2: Twice daily
E: Every 4 hours

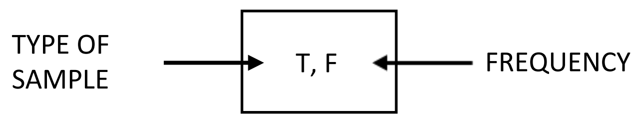


Figure C.1: Sample Laboratory Testing Program

C.2.2 Process Control Samples & Analyses

The following list contains the samples and analyses which may be required for process control within specific process streams.

Influent or Raw Wastewater

- Settleable solids.
- Total solids.
- Suspended solids.
- Volatile SS.
- Biochemical oxygen demand.
- Chemical oxygen demand.
- pH.
- Phosphates.
- Ammonia.
- Total kjeldahl nitrogen.
- Nitrates.
- Chlorides.

Grit

- Moisture content.
- Dry solids.
- Volatile solids.
- Sieve tests.

Primary Effluent

- Total solids.
- Suspended solids.
- Volatile SS.
- Biochemical oxygen demand.
- pH.
- Chemical oxygen demand.
- Total phosphate.
- Orthophosphate.

Aeration/Mixed Liquor

- Half-hour settling test of mixed liquor.
- Mixed liquor SS.
- Mixed liquor VSS.
- Sludge volume index.
- Dissolved oxygen.
- pH.
- Solids in RAS and WAS.
- Oxygen uptake rate.

Secondary Effluent

- Total solids.
- Suspended solids.
- Volatile SS.
- Biochemical oxygen demand.
- pH.
- Chemical oxygen demand.
- Total phosphate.
- Orthophosphate.

For lagoons and oxidation ponds it is most important that careful observation of the condition of the lagoon should be noted and recorded (particularly the presence of colour, algae, or odours).

Lagoon Contents

- Dissolved oxygen.
- Temperature.
- pH.

Chlorine Contact Tank
- Chlorine residual.
- Fecal coliform bacterial count.

Final Effluent
- Total solids.
- Suspended solids.
- Volatile suspended solids.
- Biochemical oxygen demand.
- Chlorine residual.
- Fecal coliform bacterial count.
- Dissolved oxygen.
- pH.
- Chemical oxygen demand.
- Total phosphate.
- Orthophosphate.
- Ammonia.

Raw Sludge
- pH.
- Dry solids.
- Volatile solids.

Waste Activated Sludge Thickening
- Solids in feed sludge.
- Solids in discharge sludge.
- Suspended solids in filtrate or centrate.
- Percent volatile in SS of filtrate or centrate.

Digested Sludge and Digester Supernatant
- pH.
- Total solids.
- Volatile solids.
- Volatile acids.
- Alkalinity.

Digester Gas
- Percent methane.
- Gas production.

Cake from Vacuum Filter or Centrifuge
- Total solids.
- Volatile solids.
- Phosphates.
- Nitrates.

Filtrate or Centrate

- pH.
- Total solids.
- Suspended solids.
- Volatile SS.

Incinerator Ash

- Dry solids.
- Volatile solids.

C.2.3 System Owner/Operator Responsibility

The System Owner shall be responsible for the sampling and analysis requirements for the proper operational control of the plant. These requirements shall ensure the proper control of day-to-day operations of the system.

C.2.4 Regulatory Authorities Responsibility

The Regulatory Authorities are only responsible for compliance enforcement. They shall not be responsible for any aspect of process control at any wastewater treatment system.

C.2.5 Sampling References

The following is a list of references which will assist operating staff in performing the necessary sampling, laboratory and control procedures to effectively operate a treatment system:

- *Standard Methods for the Examination of Water and Wastewater* from the American Public Health Association.
- *Operation of Water Resource Recovery Facilities* from the Water Environment Federation.
- *Operation of Municipal Wastewater Treatment Plants WEF Manual of Practice No.* 11 from the Water Environment Federation.
- *Field Manual For Performance Evaluation and Troubleshooting At Municipal Wastewater Treatment Facilities* from the United States Environment Protection Agency.
- *Activated Sludge Process Control and Troubleshooting* Chart from the Ohio EPA, Division of Environmental and Financial Assistance Compliance Assistance Unit.

C.3 References

Culp, G.L., Heim, N.F. *Field Manual for Performance Evaluation and Troubleshooting at Municipal Wastewater Treatment Facilities*. September 2009. <https://books.google.ca/books/about/Field_Manual_for_Performance_Evaluation.html?id=EQ60QflxM6kC&redir_esc=y>.

Ohio EPA, Division of Environmental and Financial Assistance Compliance Assistance Unit. *Activated Sludge Process Control and Troubleshooting Chart*. November 2014. <https://www.co.portage.oh.us/sites/g/files/vyhlif3706/f/pages/activated_sludge_process_control_and_troubleshooting_manual.pdf>.

Water Environment Federation. *Operation of Water Resource Recovery Facilities, MOP 11*, 7th Edition. <https://www.wef.org/wef-waterblog/wef-waterblog/new-training-resource-helps-operators-grow-professionally/>.

Water Environment Federation. *Operational of Water Resource Recovery Facilities Study Guide*. <https://www.wef.org/wef-waterblog/wef-waterblog/new-training-resource-helps-operators-grow-professionally/>.

Water Environment Federation. *Standard Methods for the Examination of Water and Wastewater*, 23rd Edition. American Public Health Association, American Water Works Association, and Water Environment Federation, 2017. <https://connect.wef.org/s/store#/store/browse/detail/a2n1S000000KVk1>.

Appendix D

Operations & Maintenance Manuals

Table of Contents

List of Tables

Appendix D

Operations & Maintenance Manuals

D.1 Use of Manuals

The purpose of an O&M manual is to give treatment system personnel the proper understanding, techniques, and references necessary to efficiently operate their facilities. The O&M manual should help to ensure the performance record of a treatment system remains high. The manual should serve as an O&M tool for plant personnel.

D.2 Preparation of Operation & Maintenance Manuals

Obtain input from experienced treatment system operators when developing O&M manuals. This input, combined with the Design Engineer's expertise, is essential to any good manual. If possible, operations input should be obtained from persons with experience in the same processes as those described in the manual.

Preparation of O&M manuals requires timely and accurate information from suppliers of wastewater treatment equipment for incorporation in O&M manuals. The information should be tailored for the specific equipment item supplied. This might include adding sections to project specifications calling for submittal of preliminary O&M information prior to paying for equipment.

Operation and maintenance manuals should possess the necessary flexibility to remain viable tools to operating personnel, in the event of changing process units or treatment system operating and maintenance needs. Operation and maintenance manuals shall contain relevant climate change threshold information, including maximum receiving water elevations, maximum flow through the plant, and other factors that cause critical conditions in the plant.

The key to an O&M manuals ultimate success is the language used and the writing style. It must be written with the end user in mind.

D.3 Operation & Maintenance Manual Checklists

The checklists presented in this section are intended to be a flexible guide for the preparation of an O&M manual for wastewater treatment systems and wastewater pumping stations and/or collection system. They can be modified to fit the particular system at hand, since project-specific requirements may be different.

D.4 Operation & Maintenance Manual Checklist for Wastewater Treatment Plants

The following checklist is adapted from that accessed March 13, 2020 at:
https://www.des.nh.gov/organization/divisions/water/wweb/documents/wastewater-facility-checklist.pdf

Table D.1: Wastewater Treatment Plan Operation & Maintenance Manual Review Checklist

	Yes	No	N/A
Table of contents.			
Chapter 1: Introduction			
1. Purpose of manual.			
a. Stand-alone manual or supplement/inclusion to an existing manual.			
2. Project description.			
a. New plant or upgraded components (list individual components if upgrade).			
b. Plant type.			
c. Simplified schematic drawing showing plant layout.			
Chapter 2: Permits & Standards			
1. Provincial permit requirements.			
a. Provincial permit full text.			
2. WSER requirements.			
a. WSER monitoring and record keeping procedures.			
b. Federal WSER reporting of non-compliance/spill procedure.			
Chapter 3: Detailed Design Criteria			
1. General description of influent wastewater.			
a. Service area.			
b. Average daily design flow.			

	Yes	No	N/A
c. Maximum daily flow.			
d. Peak hour flow.			
e. Peak instantaneous flow.			
f. Domestic flow			
g. Industrial flow.			
h. Commercial flow.			
i. Infiltration and inflow.			
j. Design BOD and TSS concentrations and loadings.			
k. Septage volumes and loads.			
l. Wastewater characterization (for nutrient removal systems).			
m. Number and location of pumping stations.			
n. Climate change threshold data (maximum design elevation of receiving water, maximum flow through the plant, and other factors that cause critical conditions in the plant).			
2. Individual unit process design criteria and physical data.			
<u>Each</u> unit process shall include the following:			
1. Influent/intermediate/effluent pumping.			
a. Wet well dimensions and volumes.			
b. Level control system.			
c. Type of pump, manufacturer, and number of units.			
d. Pump capacity L/minute at TDH.			
e. Range of flow.			
f. Horsepower (hp).			
2. Influent and effluent flow measurement.			
a. Type and manufacturer.			
b. Size.			
c. Flow range.			

	Yes	No	N/A
3. Headworks screening/comminution			
a. Type, manufacturer, and number of units.			
b. Screen size.			
c. Capacity.			
4. Grit removal.			
a. Type, manufacturer, and number of units.			
b. Tank dimensions and volumes.			
c. Type of pump, manufacturer, and number of units.			
d. Pump capacity GPM at TDH (range of flow).			
5. Septage handling.			
a. Type, manufacturer, and number of units.			
b. Tank dimensions and volumes in gallons.			
c. Pump capacity GPM at TDH (range of flow).			
d. Mixing devices.			
e. Aeration system.			
6. Primary clarification.			
a. Type, manufacturer, and number of units.			
b. Tank dimensions and volume in gallons.			
c. Weir length (each).			
d. Surface area (each).			
e. Detention times at design ADF.			
f. Surface overflow rate at design ADF and peak hour flow.			
g. Sludge pump capacity GPM at TDH (range of flow).			
h. Scum pump capacity GPM at TDH (range of flow).			
i. Sludge/scum flow measurement.			

	Yes	No	N/A
7. Secondary or advanced treatment (activated sludge, Integrated Fixed-Film Activated Sludge (IFAS), fixed film, RBC, lagoon, other).			
a. Type of process and number of units.			
b. Tank dimensions and volume in gallons.			
c. Detention time at design ADF.			
d. Biochemical oxygen demand loading.			
e. Design MLSS and MLVSS concentration.			
f. Food-to-microorganism ratio.			
g. Solids retention time.			
h. Individual anaerobic/anoxic/aerobic compartment specifications.			
i. Aeration requirements.			
j. Blowers (hp and capacity in standard L/minute.			
k. Mechanical aerators (hp and oxygen transfer rate).			
l. Mechanical mixers (hp).			
m. Recycle pumping (type, capacity GPM at TDH, and range of flow).			
8. Secondary clarification.			
a. Type, manufacturer, and number of units.			
b. Tank dimensions and volume in gallons.			
c. Surface area (each).			
d. Detention time at design ADF.			
e. Design solids loading at ADF and peak hour flow.			
f. Design surface overflow rate at ADF and peak hour flow.			
g. Weir length (each).			
h. Design weir overflow rate at ADF and peak hour flow.			
i. Return activated sludge withdrawal mechanism.			
j. Return activated sludge withdrawal rate as percent of flow.			
k. Return activated sludge pump capacity GPM at TDH, range of flow, and flow measurement.			
l. Waste activated sludge/scum pump capacity GPM at TDH, range of flow, and flow measurement.			

	Yes	No	N/A
9. Effluent filtration or other tertiary treatment.			
a. Type, manufacturer, and number of units.			
b. Design flow capacity (MGD).			
c. Design solids loading per unit.			
d. Design hydraulic loading per unit.			
e. Media surface area per unit.			
f. Tank dimensions and volumes in gallons.			
g. Backwash requirements.			
10. Disinfection (chlorination/dechlorination).			
a. Number of tanks.			
b. Tank dimensions and volumes in gallons.			
c. Detention time at peak hour flow.			
d. Point of application chemical mixing type.			
e. Chemical dose and pacing.			
f. Chemical storage tank dimensions, volumes, and containment.			
g. Chemical metering pumps.			
h. Chlorine residual monitoring.			
11. Disinfection (UV light).			
a. Type, manufacturer, and number of units.			
b. Number of bulbs/banks.			
c. Number of channels.			
d. Dose requirements.			
e. Dose pacing.			
f. Cleaning system.			
g. Transmittance/intensity monitoring.			

	Yes	No	N/A
h. Back-up disinfection alternative if UV system fails (also discussed in Chapter 4).			
i. Uninterruptable power supply			
12. Post aeration system.			
a. Type and number of units.			
b. Final effluent DO limits.			
c. Tank dimensions and volume in gallons.			
d. Air requirements.			
e. Number of diffusers.			
f. Blowers (hp and capacity in SCFM).			
g. Dissolved oxygen monitoring and pacing.			
13. Effluent disposal.			
a. Surface water/groundwater.			
b. Outfall location/GPS coordinates.			
c. Dilution factor/receiving stream water quality classification.			
d. 7Q10.			
e. Diffuser system.			
14. Plant water system.			
a. Type, manufacturer, and number of units.			
b. Capacity.			
15. Chemical feed systems for nutrient removal, solids handling, odour control, alkalinity, other.			
a. Chemical name and purpose.			
b. Storage volumes and containment.			
c. Metering pumps.			
d. Number of units.			
e. Dose pacing.			

	Yes	No	N/A
16. Odour control.			
a. Type, manufacturer, and number of units.			
b. Location of each unit.			
17. Solids handling (storage, thickening, dewatering, and stabilization).			
a. Anticipated sludge quantities.			
b. Hydraulic capacity per unit.			
c. Solids loading per unit.			
d. Performance criteria per unit.			
e. Sludge storage volumes.			
f. Sludge conveyance mechanisms.			
g. Sludge grinding mechanisms.			
h. Sludge stabilization criteria.			
18. Generator/alternate power source.			
a. Type and manufacturer.			
b. Fuel source and containment structure.			
c. Fuel storage volume.			
d. Fuel usage per hour.			
e. Run time on a full tank.			
f. Fuel storage tank location(s).			
g. List of equipment on standby power.			
19. Heating, Ventilation, and Air Conditioning (HVAC) (heating system, air handling and air conditioning units, supply and exhaust fans, unit heaters, etc.).			
a. Fuel.			
b. Capacities of each unit.			
c. Air flow/exchanges per area.			

	Yes	No	N/A
20. Fire protection and detection.			
a. Monitoring, alarms, and suppression system.			
21. Other.			
Chapter 4: Detailed Unit Process Operations & Control			
1. Plant layout schematic.			
2. Detailed process flow diagram.			
3. Hydraulic profile (include climate change design elevations and maximum flow through plant).			
4. For each unit process identified in Chapter 3, provide the following:			
a. Description and function of unit and relationship to adjacent or related units.			
b. Location of unit(s).			
c. Determination of how many units to run.			
d. Normal start-up and shut-down procedures.			
e. Normal operating conditions and control settings.			
f. Normally open/normally closed valves and gates.			
g. Unit by-pass procedure.			
h. Tank draining procedure.			
i. Anti-flotation protection for empty tanks.			
ii. Winterization and cold weather operation.			
i. Unit controls.			
i. Hand-Off-Auto (HOA) functions and switch locations.			
ii. Supervisory control and data acquisition controls.			
iii. Operator adjustable/non-adjustable set points.			
iv. Power supply.			
j. Alternate or emergency operation for equipment malfunction, process upset, and loss of power.			

	Yes	No	N/A
k. Laboratory monitoring, sampling requirements, and locations.			
l. Process control strategy.			
m. Expected unit performance.			
n. Operational problems and troubleshooting guides.			
o. High flow procedures.			
p. Operable/non-operable on generator power.			
q. Alarm conditions.			
r. Unit specific safety concerns and procedures (e.g., confined space).			
s. Unit diagrams.			
t. Unit process related formulas and example calculations.			
u. Recommended spare parts.			
v. Online monitoring systems.			
w. Digital pictures where appropriate (black and white or color).			
Chapter 5: Maintenance			
1. List of all manufacturer's O&M manuals supplied as part of this project.			
Chapter 6: Safety			
1. Health hazards.			
2. Recommended immunizations.			
3. Sewer gas dangers and confined space entry procedure.			
4. General mechanical safety.			
5. General electrical safety.			
6. Fire extinguishers/usage, locations, and maintenance.			
7. Emergency shower/eyewash stations.			
8. Recommended safety equipment.			
9. Safety data sheets for bulk chemicals used in plant.			
10. Chemical safety.			
11. Lock-out/tag out procedures.			

	Yes	No	N/A
12. Hot work permit program.			
13. Electrical arc-flash program.			
14. Automated External Defibrillator (AED) supplied equipment/location, if any.			
Chapter 7: Alarm & Notification System			
1. General description.			
2. Complete list of alarm conditions.			
3. Transmission system.			
4. After hours alarm notification and response.			
5. Routine testing of alarm systems.			
6. Loss of notification system.			
Chapter 8: Electrical Systems			
1. General description.			
2. Power distribution.			
3. Electrical system maintenance.			
4. Backup power system.			
Chapter 9: Supervisory Control and Data Acquisition System			
1. General SCADA system overview.			
2. Computer hardware.			
a. Type.			
b. Number of computers and locations.			
c. Dedicated for SCADA or multipurpose.			
d. Laptops.			
e. Remote capabilities.			
f. Maintenance and troubleshooting.			
g. Support.			

	Yes	No	N/A
3. Supervisory control and data acquisition software.			
4. Using the system.			
a. Components being monitored inclusive of pump stations.			
b. Telemetry devices.			
c. System capabilities.			
d. General operating directions.			
e. Entering set points.			
f. Alarms and alarm acknowledgement.			
g. Data archiving.			
h. Trending, graphing, and report generation.			
i. Programmable logic controllers, remote terminal, local control panels, etc.			
j. Troubleshooting guide.			
k. Glossary.			
l. Example graphics screens.			
m. System expandability.			
n. Startup procedures.			
o. Back-up power supply.			
p. Loss of phone line/transmission line (discuss back-up capabilities).			
q. Data backup capabilities.			
r. Authorization required to make changes.			
5. Supervisory control and data acquisition system security and vulnerability.			
a. Password protection.			
Chapter 10: Staffing			
1. Engineer's recommended staffing plan with supporting documentation.			
2. Classification of plant as determined by the Regulatory Authority and operator certification levels required.			

	Yes	No	N/A
Chapter 11: System Owners			
1. Contact information for all suppliers.			
2. Location of emergency shut-off valves for natural gas, propane, and water supplies.			
3. Location of main disconnect for electrical feed.			
4. Location and size of propane tanks.			
5. Location and size of fuel oil storage tanks.			
6. Communications systems (telephone, cable, radio, etc.).			
7. Location of potable water backflow devices.			
Chapter 12: Emergency Response & Contingency			
1. Site specific ERP/CP.			
Appendices			
1. Major equipment suppliers and contact information.			
2. Valve and gate schedule.			
3. Sample forms.			
a. Laboratory.			
b. Daily rounds.			
c. Process control.			
d. Solids handling.			
e. Maintenance.			
f. Provincial annual report template.			
4. Other forms as required.			

D.5 Operation & Maintenance Manual Checklist for Pumping Stations

The following checklist is adapted from that accessed March 13, 2020 at https://www.des.nh.gov/organization/divisions/water/wweb/documents/pump_station_checklist.pdf.

Table D.2: Pump Station Operation & Maintenance Manual Review Checklist

	Yes	No	N/A
Table of contents.			
Chapter 1: Introduction			
1. Purpose of manual.			
2. Use and updating information for this manual.			
3. Project description.			
a. Type, capacity, and unit processes.			
b. New or upgrade.			
c. If upgrade, describe work done and identify equipment upgraded.			
d. Collection system work, if any.			
4. Site location map.			
5. Service area.			
a. Text description.			
b. Residential, industrial, and commercial contributions.			
c. Service area map showing force mains, gravity sewers, and related pump stations.			
6. Design criteria.			
a. Average daily flow.			
b. Peak flow.			
c. Pump sizing and capacities, operating heads/inlet and outlet pressures.			
d. Wet well dimensions and capacities.			
e. Flow storage capabilities, if any.			
f. Climate change threshold data: maximum design elevation of receiving water if there is an emergency overflow, and other factors that cause critical conditions in the pumping station.			

	Yes	No	N/A
Chapter 2: System Operation & Control			
1. Identification, location and detailed description of each unit process and their relationship to each other.			
a. Screening, automatic and/or manual, bypass channel.			
b. Grinding.			
c. Grit removal.			
d. Flow measurement and calibration.			
e. Pumps.			
f. Motors.			
g. Variable-Frequency Drives (VFDs).			
h. Standby power (include a comprehensive list of what equipment is powered or not powered by stand-by power).			
i. Heating, ventilation, and air conditioning (air changes, controls, etc.).			
j. Continuous monitoring for oxygen deficiency and combustible gas (include locations of sensors and readouts).			
k. Sump pumps.			
l. Supervisory control and data acquisition or other instrumentation.			
m. Level control system (description, diagram and set points).			
n. Alarm conditions and set points for all equipment.			
o. Hoisting equipment.			
p. Odour control.			
2. Detailed operating procedures for each unit process under normal and alternate operation.			
a. Start-up and shut-down procedures/draining (include control panel graphics or pictures to illustrate).			
b. Bypassing procedures.			
c. Emergency operation.			
d. Expected unit process performance.			
e. Manual and automatic operation.			
f. Control settings.			
g. Controller locations (remote and local HOA switches, MCC panels, etc.).			

	Yes	No	N/A
3. Operational problems.			
a. Mechanical problems.			
b. Troubleshooting guides.			
c. High flow procedures.			
4. Diagrams and illustrations (no larger than 11 x 17).			
a. Piping, valve, and pump layout.			
b. Wet well layout, plans, and elevations.			
c. Alternate flow paths.			
d. Dry well layout.			
e. Valve identification and normal operational settings.			
f. Digital pictures of MCC panels or actual equipment.			
g. Instrumentation.			
5. Lab tests, if applicable.			
6. Service area collection system (if new).			
a. Layout.			
b. Cleanouts, air relief valves.			
c. Operation and maintenance.			
d. Inspection and cleaning schedule.			
e. Cleaning procedure.			
f. Identification of low-lying manholes or other areas subject to flooding or overflowing.			
Chapter 3: Maintenance			
1. Provide summaries of routine preventative maintenance activities based upon manufacturer's recommendations for each specific major piece of equipment (simply referring to the manufacturer’s O&M manual will not suffice).			
a. Lubrication schedule and type of lubricant.			
b. Special tools.			

	Yes	No	N/A
c. Valve and equipment exercising.			
d. Belt and packing replacement.			
e. Mechanical seals.			
2. Generator.			
a. Exercise under load and provide an exercise schedule.			
b. Check transfer switch.			
c. Oil and coolant specifications.			
d. Generator log with O&M records.			
3. Spare parts list (simply referring to the manufacturer's O&M manual will not suffice).			
a. Are spare parts interchangeable with other pump stations?			
4. Preventative maintenance program.			
a. Existing system.			
b. Recommended system.			
c. Equipment numbering system.			
d. Maintenance record system.			
e. Computerized maintenance management.			
f. Planning and scheduling.			
5. General maintenance practices and procedures.			
a. Mechanical maintenance.			
b. Electrical maintenance.			
6. Inventory system.			
7. Housekeeping.			

	Yes	No	N/A
Chapter 4: Personnel			
1. Personnel requirements.			
a. Staffing plan.			
b. Estimate of operational time.			
c. Frequency of visits.			
2. Job titles, job descriptions, qualifications, and experience for required positions.			
3. Training and certification.			
Chapter 5: Alarm & Notification System			
1. Summary of all alarms.			
2. Where alarms are displayed.			
3. Transmission of alarm signal to operations personnel.			
4. Periodic testing of alarm conditions and transmission devices.			
Chapter 6: Record keeping			
1. Importance of recordkeeping.			
2. Location of records.			
3. Review of recording keeping procedure.			
4. Types of records and example forms.			
a. Daily logs or station checklists.			
b. Maintenance records.			
c. Utilities records.			
i. Fuel, gas, chemical, usage, etc.			
d. Unusual events or emergency conditions.			
e. Accident reports.			

	Yes	No	N/A
5. Reporting procedures.			
Chapter 7: Safety			
1. Management and operator responsibilities.			
2. Sewer hazards.			
a. Common gases with acceptable and harmful concentrations.			
3. Mechanical hazards.			
4. Electrical hazards, including overhead wires.			
5. Chemical hazards, proper handling, and storage.			
6. Tripping and falling hazards/improper lifting.			
7. Personal hygiene.			
a. Infections.			
b. Health hazards.			
c. Immunization programs and recommended shots.			
8. Explosion and fire hazards.			
9. Road hazards and traffic control.			
10. Confined space entry procedures (one must be provided, either existing or an example).			
11. Lock-out/tag-out procedures and program.			
12. Proper housekeeping.			
13. Safety data sheets for bulk chemicals.			
14. List of recommended and existing safety equipment.			
15. Training.			
16. Safety reference library.			

	Yes	No	N/A
Chapter 8: Emergency Operating Plans & Procedures			
1. Vulnerability analysis for the following emergency conditions.			
a. Power failure.			
b. Equipment failure.			
c. Natural disasters.			
i. Flooding.			
ii. Hurricane or strong winds.			
iii. Earthquake.			
iv. Freezing conditions.			
d. Hydraulic overloading.			
i. Identify low lying manholes or other areas of concern and provide elevations.			
ii. Provide locations of nearby wells or surface waters.			
e. Ruptures.			
f. Bypassing options.			
i. Upstream/downstream manholes.			
ii. Emergency pumping connections.			
g. Sewer blockages.			
h. Spills of oils, toxic, or hazardous materials into the sewer system or at the pump station.			
i. Explosion.			
j. Fire.			
k. Failure of emergency warning system.			
l. Labour strikes.			
m. Personnel injury.			
n. Other emergency situations.			
2. Methods to reduce vulnerability.			
3. Emergency response for each condition.			
4. Follow-up investigation and prevention plan.			

	Yes	No	N/A
5. Sewer overflow reporting procedure (provincial and federal).			
6. Emergency notification system.			
7. Notification of downstream water users.			
8. Complete emergency contact telephone list.			
a. Provincial agencies.			
b. Town or city officials.			
c. Provincial emergency services (police, fire department, etc.).			
d. Chemical spill response units.			
e. Hazardous waste/oil spill cleanup firms.			
f. Local hospitals.			
g. Emergency pumping equipment suppliers.			
h. Emergency power equipment suppliers.			
i. System Owner providers.			
j. General contractors.			
k. Septage hauling firms.			
l. Electricians.			
m. Supervisory control and data acquisition technicians.			
n. Downstream water users.			
9. Emergency equipment inventory and location of equipment.			
10. Personnel training and interaction with local emergency response entities.			
Chapter 9: Utilities			
1. Suppliers and contact information for all utilities.			
a. Electrical.			
b. Gas, propane, fuel, oil.			
c. Water.			

	Yes	No	N/A
d. Telephone.			
e. Alarm communications/SCADA.			
2. Provide exact locations of emergency shut-off valves, backflow preventers, etc.			
3. Provide sizes and locations of bulk storage tanks.			
4. Provide a Spill Prevention Containment and Control Plan for bulk storage tanks.			
Chapter 10: Electrical & Control Systems			
1. General description of electrical and control system.			
2. Describe MCC panels including schematics or simple drawings.			
Chapter 11: SCADA (if applicable)			
1. Detailed description including SCADA graphics.			
Appendices			
1. Schematics and flow diagrams showing all pertinent equipment and major piping (11 x 17 max).			
2. Schematic of collection system, if applicable.			
3. Detailed design criteria.			
4. Sample forms including daily operational checklists.			
5. Piping color codes.			
6. Equipment supplier’s information.			
7. List of all manufacturer’s manuals.			
8. Other pertinent information.			
9. Supervisory control and data acquisition graphics overview.			

Appendix E

Biosolids Utilization on Land

Table of Contents

List of Tables

Appendix E

Biosolids Utilization on Land

E.1 Biosolids Utilization on Land

These Guidelines were formulated to provide the minimum criteria for municipal treated sludge or biosolids and its utilization on land, where applicable. Wastewater biosolids can be useful to crops and soils by providing nutrients and organic matter. System Owners considering land application should discuss their plans in advance with provincial Regulatory Authorities to determine whether their sludge quality is appropriate for land application.

E.2 Biosolids Quality Criteria

E.2.1 Supervisory Control & Data Acquisition

Biosolids quality is determined by the pathogen and metal content and is dependent on the wastewater characteristics and the type of treatment. Biosolids acceptable for land application and/or storage fall in to one of three categories, depending on the metal and pathogen content:

1. Exceptional quality.
2. Class A.
3. Class B.

There are no restrictions for land application of EQ Biosolids or biosolids regulated under the *Fertilizers Act* from the Government of Canada. Land application of Class A or Class B biosolids typically requires an approval from Regulatory Authorities, and restrictions pertaining to the use of these products will apply.

E.2.2 Metals

All biosolids contain variable amounts of metals, some of which are essential plant nutrients (micronutrients). When applied to soils in excessive amounts, metals may accumulate in soils. Soil loadings of metals must, therefore, be controlled in biosolids application. The metal concentration in biosolids intended for land application (EQ or Class A/Class B) must not exceed the maximum acceptable metal concentrations in Table E.1.

Some jurisdictions may have more stringent guidelines for disposal of biosolids (i.e., *Guidelines for Land Application and Storage of Biosolids in Nova Scotia* from Nova Scotia Environment).

Table E.1: Maximum Acceptable Concentrations in Biosolids (mg/kg of Dry Weight)

Metal	Exceptional Quality	Class A/Class B
Arsenic	41	75
Cadmium	39	85
Chromium	1,200	3,000
Copper	1,500	4,300
Mercury	17	57
Molybdenum		75
Nickel	420	420
Lead	300	840
Selenium	36	100
Zinc	2,800	7,500

Adapted from *Biosolids Applied to Land: Advancing Standards and Practices* from the United States Environment Protection Agency (2002).

E.2.3 Sludge Stabilization

Only stabilized wastewater sludge (biosolids) should be applied to land. Biosolids are defined as processed sludge in which the organic and bacterial contents of raw sludge are reduced to levels deemed necessary by the Regulatory Authority to reduce nuisance odours, pathogen concentration, vector attraction, and public health hazards.

Biosolids may be defined as stabilized if one of the following conditions can be met:

- Volatile solids in the sludge have been reduced to at least 50% of TS.

The percentage reduction can be calculated using the *VanKleek Formula* below:

$$\%\,Reduction = 100\ \times \frac{\%\,Vsin - \%\,Vsout}{\%\,Vsin - (\%\,Vsin \times \%\,Vsout)}$$

- The Specific Oxygen Uptake Rate (SOUR) of the sludge is less than 1.5 mg O_2/hour/g of total sludge on a dry weight basis corrected to 20°C. This test is only applicable to liquid aerobic biosolids withdrawn from an aerobic process.
- Sludge meets the high pH stabilization criteria described in Section 12.7.3.
- Any process which produces sludge equivalent in quality to the above in terms of public health factors and odour potential may be accepted. Additional treatment would be required to further reduce pathogens when the sludge is to be spread on dairy pastures and other crops which are in the human food chain. Biosolids generators are responsible for the stabilization and verification of any biosolids intended for land application. System Owners must provide sufficient information acceptable to demonstrate that the biosolids have been effectively stabilized to meet pathogen reduction requirements.

E.2.4 Pathogens

Pathogens are disease causing organisms such as bacteria, viruses, and parasites that exist in all biosolids. The pathogen reduction requirements for each of the three categories of biosolids are listed in Table E.2.

Table E.2: Pathogen Reduction Requirements

Exceptional Quality	Class A	Class B
Fecal coliform: < 1,000 MPN/g TS (dry weight) OR Salmonella: < 3 MPN/4 g Total solids (dry weight)	Fecal coliform: < 1,000 MPN/g TS (dry weight) OR Salmonella: < 3 MPN/4 g Total solids (dry weight)	Fecal coliform: < 2,000,000 MPN/g of TS (dry weight)

E.2.5 Persistent Organic Chemicals

Sufficient information is not available to establish criteria of sludge spreading in regard to persistent organic chemicals, such as pesticides and Polychlorinated Biphenyls (PCB), however, if there is a known source in the wastewater system service area which discharges or discharged in the past such chemicals, the sludge should be analyzed for such chemicals and the Regulatory Authority should be consulted for recommendations concerning sludge spreading.

E.3 Site Selection

E.3.1 General

By proper selection of the biosolid application site, the nuisance potential and public health hazard should be minimized. The following items should be considered, and the Regulatory Authority should be consulted for specific limits:

- Land ownership information.
- Groundwater table and depth to bedrock location.
- Location of dwellings, roads, and public access.
- Location of wells, springs, creeks, streams, and flood plains.
- Slope of land surface.
- Soil characteristics.
- Climatological information (including climate change projections).
- Land use plan.
- Road weight restrictions.

E.3.2 Site Location

The following restrictions should apply to the location of a proposed biosolid to land application site:

- The site should be remote from surface water courses. The minimum distance between the site and the high-water mark of the surface water course should be determined by the land slope as presented in Table E.3.

Table E.3: Minimum Distance to Watercourse

Maximum Sustained Slope	For Biosolid Application (May to November Inclusive)	For Biosolid Application (December to April Inclusive)
0 - 3%	100 m	No biosolids to be applied.
3 - 6%	125 m	No biosolids to be applied.
6 - 8%	180 m	No biosolids to be applied.
Greater than 8%	No biosolid to be applied unless special conditions exist.	No biosolids to be applied.

For groundwater separation distances See Table E.4.

- No processed organic waste should be applied to the site during any period in which conditions are such that surface runoff is likely to occur considering land slope, soil permeability, and the climatic conditions of the area.
- The site should be located a minimum distance from certain physical features, as specified in Table E.3.

Table E.4: Minimum Distance to Physical Features

Type of Feature	Minimum Setback Distance
Public wells	150 m**
Private wells	90 m**
Property line	10 m*
Bedrock outcrops	10 m*
Dwellings	90 m**
Institutional buildings (e.g., schools and hospitals)	200 m**
Commercial buildings	90 m
Uninhabited buildings	30 m
Public areas (e.g., parks and playgrounds)	90 m
Perennial water bodies and watercourses	100 m
Intermittent water bodies and watercourses	100 m
Swales and man-made drainage ditches	15 m
Primary and secondary roads	30 m*
Unimproved dirt roads	10 m*

Note: *100 m setback required for spray irrigation areas.
**300 m setback required for storage lagoons and spray irrigation areas.

- Berms and dykes of low permeability should be constructed on the site where necessary to isolate the site and effectively prevent the egress of contaminants.
- No biosolids application should be located on a flood plain, an area which is inundated by a flood that has a 1% or greater chance in recurring in any year, or a flood of a magnitude equaled or exceeded 1-in-100-years on average. Additional controls are required when biosolids are applied in flood risk areas. A flood risk area is a flat or gently sloping area beside a watercourse which may be subjected to flooding. The land application of both Class A and Class B biosolids in a flood risk area, which may experience flooding once in 20 years, must not occur before the risk of flood has passed, any flood waters have returned to their normal level, and the soil is adequately drained to support application equipment. Class A and Class B biosolids applied to land in flood risk areas must be directly injected into the soil or surface applied followed by incorporation (within 24 hours of spreading). The storage of Class A or Class B biosolids is not permitted in a flood risk area which may experience flooding 1-in-100-years.
- No wastewater biosolids application sites should be installed within the area of any municipal watershed unless permission is granted by the Regulatory Authority having jurisdiction.
- A wastewater biosolids application sites should not be located over land areas with a seasonal high-water table at less than 450 mm below the ground surface, or with bedrock at less than 900 mm.

E.3.3 Land Characteristics

The following restrictions should apply to the land characteristics of a proposed sludge to land application site:

- The land slope and soil permeability will determine the time of year that sludge may be applied.
- The groundwater table during sludge application should be not less than 1.0 m from the surface for soils with moderate to slow permeability. For soils with rapid to moderately rapid permeability the groundwater table should be not less than 1.5 m from the surface.
- Where sludge application is carried out by tank truck, untiled land should be given preference to tiled land. Where tiled land is used the sludge hauling contractor should request instructions from the landowner, with regards to minimizing the possibility of damage to the tile system.

Table E.5: Sludge Application Periods

Maximum Sustained Slope	Soil Permeability	Allowable Duration of Application
0 - 3%	Any	7 months/year (May to November)
3 - 6%	(> 5×10^{-5} - 8×10^{-6} m/s) Rapid to moderately rapid	7 months/year (May to November)
	(2×10^{-6} - 5×10^{-7} m/s) Moderate to slow	6 months/year (May to October)
6 - 8%	(>5×10^{-5} - 8×10^{-6} m/s) Rapid to moderately rapid	7 months/year (May to November)
	(2×10^{-6} - 5×10^{-7} m/s) Moderate to slow	6 months/year (May to October)
Greater than 8%	Any	No sludge applications unless warranted by special conditions

E.3.4 Minimum pH

No biosolid should be applied on land if the soil pH is less than 6.0 at the time the biosolid is applied. Soils intended for biosolids application must have a pH between 6.0 and 8.0 to minimize metal leaching. Alkaline stabilized sludges may be applied to soils of lower pH when they raise the soil pH to at least 6.0. The soil pH should be maintained between 6.0 and 8.0 for at least 2 years following the end of biosolids application.

E.3.5 Land Use Restrictions & Waiting Periods

Sludge should not be applied to land which is used for growing food crops to be eaten for human consumption.

Land on which Class B biosolids have been applied must adhere to the waiting periods identified in Table E.6. Class A and Class B biosolids are not permitted for use on residential lawns or gardens.

Table E.6: Minimum Waiting Periods

Land Use	Waiting Period
Pasture	Not in the same calendar year
Forage	2 months before harvest
Livestock feed	2 months before harvest
Food crops (edible parts below soil surface)	38 months before harvest
Food crops (edible parts above soil surface)	18 months before harvest
Commercial sod	12 months before harvest

Application sites where Class A and Class B biosolids have been applied should have required appropriate signage to identify the site as having received biosolids. Signs must be placed at all four corners of the application site as well as on each access road or path to the site. For Class B biosolids signs must remain in place for 38 months following the most recent application. Application sites where Class A biosolids have been applied, temporary signage (2 months) is required. Typical signage should include the following wording:

Biosolids Application Site
System Owner Name
Identify Biosolids Source(s)
Field No. 1; NE Corner

The signage must be maintained so that it remains in place and can be easily read for the required time period.

E.4 Application Rate & Methodology

E.4.1 Nutrient & Land Management Plans

Land application of biosolids, when pertaining to agricultural land, should follow a NMP or, when pertaining to land other than that used for agricultural purposes (e.g., reclamation sites), should follow a Land Application Plan (LAP).

Nutrient Management Plans should be prepared by Nutrient Management Planners and should outline crop requirements and biosolids parameters. The NMP should determine the biosolids application rate based on the agronomic rate. Biosolids should be applied as close to the time of maximum nutrient uptake of crops as feasible. The application rate should ensure that metal concentrations in soils do not exceed the limits specified in Table E.7.

Land application plans should be prepared by a professional engineer or agrologist. The LAP should outline crop/vegetation requirements and biosolids parameters and should determine the biosolids application rate based on nutrient and organic matter requirements. The rate of application should ensure that the appropriate amount of nutrients is applied to the soil to prevent groundwater contamination. The application rate should ensure that metal concentrations in soils do not exceed the limits specified in Table E.7.

E.4.2 Acceptable Application Methods

With the exception of flood risk areas, Class A biosolids may be land applied by surface spreading as a top dressing, through incorporation, or by injection below the surface of the soil or as permitted by the Regulatory Authority having jurisdiction. Class B biosolids may be surface spread followed by incorporation or may be injected below the surface of the soil. For Class B biosolids, incorporation must take place within 24 hours of spreading.

For Class A and Class B biosolids, land application is not permitted when the ground is frozen, snow covered, or saturated. Biosolids must not be applied to land during or immediately following heavy rains or when heavy precipitation is forecasted, which may adversely affect the environment, through surface water run-off, and/or the ability to effectively spread and incorporate the biosolids on the field(s).

E.4.3 Additive Metal Loading Restrictions

Unrestricted addition of metals to agricultural soils will result in both elevated metal content of the crops and plant toxicity. The following restrictions (with a built-in safety factor) are designed to control this potential problem.

Table E.7 lists criteria for the metal content in soils.

Table E.7: Criteria for Metal Content in Soils

Metal	Maximum Acceptable Metal Addition to Soil (kg/Ha)	Maximum Acceptable Metal Content in Sludged Soils (µg/g)
As	14	14
Cd	1.6	1.6
Co	30	20
Cr	210	120
Cu	150	100
Hg	0.8	0.5
Mo	4	4
Ni	32	32
Pb	90	60
Se	2.4	1.6
Zn	330	220

E.5 Sludge Application on Forested Land

Disposal of sludge on forested land is considerably less hazardous than on cropland in terms of heavy metal toxicity unless the land is to be converted to cropland. For the allowable sludge loading the Regulatory Authority should be consulted.

E.6 Management of Spreading Operation

E.6.1 Hauling Equipment

The sludge hauling equipment should be designed to prevent spillage, odour, and other public nuisances.

E.6.2 Valve Control

The spreading tank truck should be provided with a control so that the discharge valve can be opened and closed by the driver while the vehicle is in motion. The spreading valve should be of the "fail-safe" type (e.g., self-closing) or an additional manual stand-by valve should be employed to prevent uncontrolled spreading or spillage.

E.6.3 Biosolids Storage

Sufficient storage capacity should be provided for periods of inclement weather and equipment failure. The storage facilities should be designed, located, and operated to avoid nuisance conditions. See Section 12.5.2 for more information regarding biosolids storage.

E.6.4 Spreading Methods

The selection of spreading methods depends on the sludge characteristics, environmental factors, and others. When control of odour nuisance and runoff is required, immediate incorporation of sludge after spreading or subsurface injection should be considered. When such a method is utilized, an adjustment in the reduced rate of ammonia loss into the atmosphere should be considered in the computation for nitrogen balance.

The sludge should be spread uniformly over the surface when tank truck spreading, ridge and furrow irrigation or other methods are used. Wastewater sludge application should not be made during or immediately after rainfall.

Proposals for subsurface application of sludge should include for review a description of the equipment program for application.

Spray systems, except for downward directed types, will not ordinarily be approved.

E.6.5 Monitoring, Reporting, & Record Keeping

The requirements of the Regulatory Authority on the monitoring, reporting, and record keeping of the biosolids spreading operation should be followed. As a minimum, the producer of sludge should regularly collect and record information on the biosolids and soil characteristics and the volume of biosolids spread on a particular site.

E.7 Biosolids Storage

E.7.1 Biosolids Storage

The storage of biosolids may be required at times when land application is not possible and sufficient storage should be available to retain biosolids during these circumstances. The storage of Class A or Class B biosolids at land application sites must be approved in writing by the Regulatory Authority.

Class A and Class B biosolids with a minimum solids content of 20% may be stockpiled, or stored temporarily, at the application site prior to land application, provided that the biosolids are intended for use at that location. Biosolids can be stockpiled without an impermeable surface for up to 1 week at the application site prior to land application, unless otherwise approved in writing by the Regulatory Authority. Stockpiled biosolids must be fully covered with an impermeable material, such as a tarp. Stockpiles must be located to minimize contact with surface water run-off and to prevent infiltration of precipitation and the generation of leachate. Class A and Class B biosolids with a minimum solids content of 20% may be stored for more than one week on top of an impermeable surface such as a concrete pad or clay liner at the application site prior to land application. The impermeable surface should have curbed sidewalls or berms on all sides constructed of the same material. Clay liners should have a minimum thickness of 0.5 m and an in-situ coefficient of permeability of 1.3×10^{-6} cm/s. Such biosolids storage areas should be fully covered with an impermeable material, such as a tarp. Stockpiled biosolids should be fully covered with an impermeable material, such as a tarp. In addition, such storage areas must be located to minimize contact with surface water run-off and to prevent infiltration of precipitation and the generation of leachate.

The storage of Class A and Class B biosolids with a solids content of less than 20% must be in lagoons only. Storage lagoons must be designed by a professional engineer. Biosolids may be stored temporarily (storage of less than 72 hours) in a tank approved by the Regulatory Authority on land application sites.

E.7.2 Volume

Rational calculations justifying the number of days of storage to be provided should be submitted and should be based on the total sludge handling and disposal system. Sludge production values for stabilization processes should be justified in the basis of design. If the land application method of sludge disposal is the only means of disposal utilized at a treatment plant, storage should be provided based on considerations including at least the following items:

- Inclement weather effects on access to the application land.
- Temperatures including frozen ground and stored sludge cake conditions.
- Haul road restrictions including spring thawing conditions.
- Area seasonal rainfall patterns.
- Cropping practices on available land.
- Potential for increased sludge volumes from industrial sources during the design life of the plant.
- Available area for expanding sludge storage.
- Climate change parameters affecting any of the above items.

A minimum range of 120 to 180 days storage should be provided for the design life of the plant unless a different period is approved by the Regulatory Authority.

E.8 References

Canadian Food Inspection Agency. *Fertilizers Act*. Government of Canada, 2020. <https://laws-lois.justice.gc.ca/eng/acts/f-10/>

Nova Scotia Environment and Labour. *Guidelines for Land Application and Storage of Biosolids in Nova Scotia*. Nova Scotia Environment, 2004.

U.S. Environmental Protection Agency. *Biosolids Applied to Land: Advancing Standards and Practices*. Washington, DC: USEPA, 2002.

Appendix F

Factors for Choosing Climate Change Information

Table of Contents

Appendix F

Factors for Choosing Climate Change Information

In this Appendix, many of the concepts discussed in Chapter 1 are applied specifically to choosing climate information. A list of factors that drive the selection of climate information is presented in this Appendix.

F.1 Asset – Climate Interactions

A major factor for choosing climate information are the asset-climate interactions:

- **Interactions**: the asset-climate interaction(s) (i.e., impacts) being assessed are what drive the need for specific climate information. Which variables, and what aspects of the variables or indices are relevant, depends on the how the asset or program has been or will be impacted by climate change. The list of relevant interactions depends on **what is being impacted** (the asset or program). For example, a yield study likely needs to examine dry weather, whereas a water quality study may be concerned with high-intensity precipitation. Furthermore, wastewater discharge limits described in the plant's Approval/License/Permit to Operate would consider both dry and wet weather flows. While the 1-in-50-year wind is relevant for buildings, changes in average monthly winds are not directly applicable. The list of relevant interactions also depend on what is **causing the impact** (climate change). Such as the geographic context, topography, proximity to the coast, microclimate, vegetation, geology, etc. For example, permafrost may need to be considered in Labrador, but is not applicable in Prince Edward Island.

F.2 Assessment

The following are factors for choosing climate information based on the characteristics of assessment:

- **Budget/Timeline**: in some cases, compromises are made due to the assessments' practical constraints. For example, while a yield assessment would ideally be informed by hydrological or hydrogeological impact modelling, trends can be approximated, with caveats, from precipitation information.
- **Type of Input Required**: the assessment type may dictate the format or other characteristics of the climate information required. For example, flood modelling that is based on the 1-in-100-year storm requires climate information on the projections for future precipitation rates (e.g., increase from 200 mm by 30% to 260 mm), whereas a risk assessment based on the probability of the existing 1-in-100-year storm requires climate information on the projected increased frequency of such an event (e.g., 0.01 annual exceedance probability becomes 0.015).

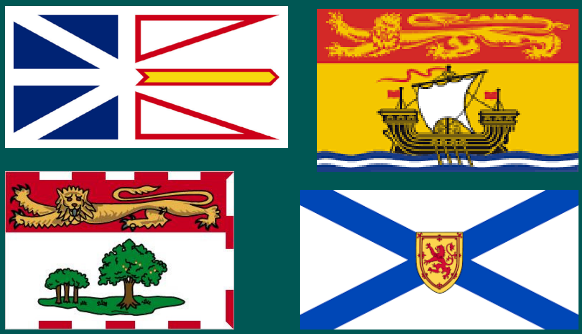
- **Projection Horizons**: as described in Section 2.2, projection horizons or timelines are chosen based on the remaining useful life of the program or asset being assessed. Sources of uncertainty are relevant to different timescales, which means that different types of climate information may be required depending on the projection horizon. For short projection horizons (e.g., 10-year lead times), the natural climate variability may dominate and can even hide the climate change signal. For that reason, climate risk in these time periods needs to consider variability based on historical measured information or certain types of climate model ensembles.
- **Risk Tolerance and Uncertainty Characterization**: infrastructure types have different risk tolerances. For a dam, a risk assessor may want to understand how the 95th percentile of climate model projections for drought may affect desiccation cracking in the structure. The best practice to characterize uncertainty is to use more than one emissions scenario and a range of projections from an ensemble of climate models. For some variables, more than one projection method is also warranted. Using the median of a modelling ensemble, however, ignores the potential risk of higher or lower projections. This may be sufficient for a non-critical asset, with a higher risk tolerance than a critical asset.
- **Precision Level:** assessments vary widely in the degree of precision required. Some risk assessments are more qualitative, and they require a general trend or degree of change to assign climate changes as low, medium, high, or a binned score from 0 to 5. The anticipated impact on results also dictates the precision level required for certain variables. For example, a 5˚C change in water temperature may be irrelevant to the planned design, in which case literature may be sufficient.
- **Geographic Scale**: whether an assessment is conducted for a single locality, or at a larger scale (e.g., province-scale) will influence the type of climate information that is relevant. There is also a practical aspect, for example, obtaining gridded projection data may be warranted for a larger assessment in order to visualize geographic variability in projections.

F.3 Climate Information

The following are factors for choosing climate information based on the characteristics of climate information sources:

- **Source Recognition**: only sources of climate information vetted by experts should be used, such as Environment and Climate Change Canada. It is important that the methodologies have been well validated (tested) and that metadata is available.
- **Methodology Assumptions**: sources of information should be used within the confines of their assumptions. For example, the *IDF_CC Tool* from the Western University Canada assumes that the relationship between daily and sub-daily precipitation is unchanged in the future, and the method should only be used in conjunction with other approaches.
- **Updated Methodology**: climate science moves at a fast pace. Emissions scenarios, modelling ensembles, and post-processing techniques are regularly updated and improved. Unless it cannot be avoided, it is important not to use sources of climate information that have been superseded. Using older information could mean it is outdated and will have an impact on the design process.
- **Available Variables**: the variables required for an assessment will dictate the sources of climate information. Temperature and precipitation are the two variables which are the are most readily available from climate models. That said, projections for changes in frequency and/or intensity of hurricanes are better obtained from literature.
- **Available Indices**: only certain indices are available from data portals and past literature. If an assessment requires a specific infrastructure-related threshold, it may be necessary to calculate the projection by post-processing daily projection timeseries.

- **Available Ensemble**: sources of information must have a sufficient ensemble size. It is inadequate to use projections from only a few climate models.
- **Uncertainty Characterization**: not all sources of climate information Report on the range of possible projections (e.g., percentiles). For example, literature for sea-level rise projections may report only the median of the model ensemble. Especially for assessments with a low risk tolerance, it is important to obtain projections where uncertainty characterization is also available.
- **Resolution**: not all climate information sources will provide the spatial and temporal resolution needed for an assessment. For example, the ensemble of downscaled GCMs on ClimateData.ca has daily projections only, and it cannot inform sub-daily precipitation extremes.
- **Projection Horizon**: not all sources are available for all projection horizons. Some indices from data portals may have been pre-calculated on certain tri-decades. Historical sources of information obviously do not provide information on future projection periods.



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