Guidelines for the Design, Construction and Operation of Water and Sewerage Systems

Government of Newfoundland and Labrador
Department of Environment and Conservation
Water Resources Management Division

December 2005

Foreword

The Water Resources Management Division of the Department of Environment and Conservation considers the Guidelines for the Design, Construction and Operation of Water and Sewerage Systems as an integral part of its regulatory program directed at ensuring public health and environmental protection.

This document supersedes April 1980 version of the "Guidelines for the Design, Construction and Operation of Water and Sewerage Systems". During the preparation of the current document, the main objectives were: (i) to keep pace with the new development in water and sewer industry and provide appropriate guidance and (ii) to ensure the consistency between the content of guidelines document and the master specifications document. The document provides general guidance on good engineering practices for design, construction, operation and maintenance aspects of water and sewerage systems.

As a part of the review process, the Water Resources Management Division referred and reviewed: the Nova Scotia Department of Environment Standards and Guidelines Manual For The Collection, Treatment and Disposal of Sanitary Sewage; the Alberta Environmental Protection Standards and Guidelines For Municipal Waterworks, Wastewater and Storm Drainage Systems; the Atlantic Canada Standards and Guidelines Manual For The Collection, Treatment and Disposal of Sanitary Sewage; the Great Lakes-Upper Mississippi River Board of State Public Health & Environmental Managers Recommended Standards For Water Works and the Recommended Standards For Wastewater Facilities; the Government of Newfoundland and Labrador Municipal Water, Sewer and Road Specifications, and et al. We also provided the earlier draft of the document to appropriate government departments, municipal staff, builders and contractors, and consultants, for their review and comments. The review comments have improved the quality of the document and reviewer's contribution is very much appreciated.

I would like to extend special thanks to Deneen Spracklin for coordinating the overall completion of this project. I would also like to take this opportunity to acknowledge the work of Ron Goulding, Herbert Card, Erik Neilson and Chris Blanchard in revising and re-writing the various sections of this document, and thank them for the excellent contribution during numerous project meetings.

The previous version of this document was shared with the Water Resources Management Division, Department of Municipal Affairs, Department of Government Services, and Consulting Engineers. The staff of these agencies provided valuable comments which have been incorporated in this version.

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Manager

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Section 1 – Approval by Regulatory Authority

1. Approval by Regulatory Authority

Early consultation with the Department of Environment and Conservation (DOEC) is recommended when contemplating the development of a new water supply, wastewater treatment, expansion or upgrading of existing systems. DOEC staff will assist, give direction and help the designer to meet the regulatory requirements, and thereby avoid time delays and costly revisions later in the review and permitting phases.

1.1. Submission

All plans and specifications of the works to be undertaken shall be submitted for approval to the DOEC, before any construction work may commence. The submission must be in accordance with the requirements of the *Water Resources Act*, SNL 2002 cW-4.01.

The submission shall be accompanied by an official application form duly completed and signed where applicable, by the Engineer acting for the Owner/Proponent, the Owner/Proponent themselves, President of the Corporation, or other properly authorized representative of the executive group of the corporation, municipality or institution and the appropriate fee.

All items of the submission shall bear the stamp and signature of an Engineer registered in the Province of Newfoundland and Labrador.

1.2. Review Period

A minimum period of thirty working days, from the date of receipt by the DOEC, shall be allowed for the review of the submission.

1.3. Permits

If the submission meets the requirements of the DOEC, a Permit to Construct will be issued to the Owner/Proponent by the DOEC. No changes shall be made to the approved documents without the formal approval of the DOEC.

The DOEC will also issue a separate Permit to Operate for all water and sewerage works.

1.4. Payment

In accordance with Section 65 of the *Water Resources Act*, SNL 2002, cW-4.01, an application fee(s) must be paid to obtain a permit as required under Sections 36 and 37 of the *Water Resources Act*, SNL 2002, cW-4.01.

1.5. Limits of Review and Approval

Generally speaking, the DOEC shall review and approve the following submission details:

1. Functional and sanitary features;
2. Location and layout of structures;
3. Process design;
4. Equipment;
5. Control features; and
6. Instrumentation.

It is not the responsibility of the DOEC to make a detailed check of the structural design, therefore this aspect of the design is not included in the Permit to Construct.

The DOEC recognizes advancements in research and development of innovative technologies and encourages designers to incorporate state of the art technology in their submissions when demonstrated advantage, improvement of operation, or superior performance is evident.

The DOEC shall base its Permit to Construct upon design data as submitted by the Design Engineer, and the DOEC take no responsibility for the correctness of such data.

Section 2 – Project Reports

2. Project Reports

2.1. Preliminary Engineering Report

All water and sewerage system projects require the approval of the Department. Therefore it is essential that the general acceptability of a project, to the Department, be identified prior to detailed design being started.

Depending upon the size and complexity of a project, this would be achieved by either, or a combination, of the following:

1. Meeting(s) between the Owner/Proponent and/or consulting engineer and the Department;
2. Submission of a Conceptual Report; and
3. Submission of a Pre-design Report (Feasibility Report, Preliminary Engineering Report).

The Conceptual Report would only be required for more complex projects involving several possible alternative solutions.

2.2. Technical Report

A Technical Report could be in the form of a pre-design report, preliminary report, or any other document approved by the Department.

The purpose of a Technical Report is to investigate the design alternatives in sufficient detail to permit their evaluation with respect to capital and operating costs, the extent to which they resolve the problem and their technical feasibility in terms of accepted engineering practices, and the potential environmental consequences of their implementation. In investigating the alternative solutions, current and historical information should be reviewed with respect to:

Water System

1. Existing water demand at the treatment facility;
2. Raw water quality;
3. Operational problems;
4. System design parameters;
5. Condition of the treatment and distribution facilities;
6. Planned development in the community; and
7. Adjacent land uses.

Sewerage System

1. Existing sewage loads at the treatment facility;
2. Raw sewage and effluent quality;
3. Operational problems;
4. System design parameters;
5. Condition of the collection and treatment facilities;
6. Planned development in the community; and
7. Adjacent land uses.

2.2.1. Population

It is extremely important to finalize the design population as early as possible in the Pre-design process. The acceptability of the proposed population should be discussed with all appropriate agencies. Population data should be accessed for as long as records show to provide trending possibilities.

2.2.2. General Considerations

All necessary information should be collected and examined including:

1. Description of the existing water works and sewerage facilities;
2. Description of the nature and extent of area to be served;
3. Soil data, e.g., depth of overburden, soils structural properties;
4. Topography, e.g., slope and floodplain mapping;
5. Sewage flow records;
6. Water usage records, including fire fighting requirements;
7. Influent/effluent quality records;
8. Data on specifics of operational/treatment problems occurring at the treatment facility, where applicable;
9. Maintenance activity records;
10. Provisions for extending the water and sewerage systems to additional areas;
11. Appraisal for the future requirements for service, including existing and industrial, commercial, institutional and other water and sewerage system needs; and
12. Treatment requirements to include recommendations on technologies and future expansion.

Items that should be addressed for specific problems include the following:

1. Well log data, e.g., well locations, water table levels, groundwater quality, soil materials;
2. Water intake location - surveys, soundings and sampling for analysis;
3. Water quality and flow records for receiving water body or surface water source;
4. Data on water usage, e.g., upstream, downstream treatment facilities, recreational use; and
5. Location, size, operational characteristics of adjacent individual/communal wells and subsurface disposal systems, where applicable.

2.2.3. Ocean Outfalls

The Technical Report should include, as a minimum, the following information:

1. Route of ocean currents in the vicinity of the proposed outfall;
2. Location of all fishery, marine resources and recreational activity in the vicinity;
3. Soundings in vicinity of outfall;
4. Effect of winds and tides on the receiving waters; and
5. Dispersion characteristics.

2.2.4. Industrial, Institutional or Commercial Wastes

Where possible, industrial, institutional or commercial water discharging to the sewerage system should be identified. In addition, the potential impact of these discharges on the treatment facility should be highlighted. Where applicable, treatment of the waste prior to discharge to the sewerage system should be considered.

2.2.5. Regulations

Reference should be made to the *Environmental Control Water and Sewage Regulations* regarding the allowable discharges to a sanitary sewer and from a sewage treatment plant.

Section 3 – Water Works

3. Water Works

3.1. General

The Engineer should confer with the DOEC before proceeding with the design of any water works. Certain basic requirements are mandatory and others shall be considered to be good practice.

A Technical Report prepared and signed by the Engineer representing the Owner must be submitted to the DOEC. The Technical Report is to be completed in accordance with Section 2 of this Document.

3.1.1. Plant Layout

The following items should be considered with regards to plant layout:

1. Functional aspects of the plant layout,
2. Provisions for future plant expansion,
3. Provisions for expansion of the plant's waste treatment and disposal facilities,
4. Access roads,
5. Site grading,
6. Site drainage,
7. Walks,
8. Driveways, and
9. Chemical delivery.

3.1.2. Building Layout

Design of the building shall provide for:

1. Adequate ventilation, lighting, heating, and drainage,
2. Dehumidification equipment, if needed,
3. Accessibility of equipment for operation, servicing and removal/replacement,
4. Flexibility of operation,
5. Operator and visitor safety,
6. Convenience of operation, and
7. Separation of chemical storage and feed equipment areas to reduce hazards and dust problems.

3.1.3. Location of Structures

Structures shall not be located in areas subject to flooding nor shall impede normal or flood stream flows.

3.1.4. Electrical Controls

Main switch and electrical control gear shall be located above grade, in areas not subject to flooding. Surge protection should be provided.

3.1.5. Standby Power

Dedicated auxiliary standby power shall be required so that the water may be treated and/or pumped to the distribution system during power outages to meet at least the average day demand.

3.1.6. Shop Space and Storage

Shop space and storage consistent with the facilities as designed shall be provided.

3.1.7. Laboratory Equipment

Laboratory equipment and facilities shall be selected and designed to complete in-plant analysis and routine laboratory testing as may be determined by the quality of the raw water source, the complexity of the treatment process, and the extent of the water distribution system involved. The design engineer shall confer with the DOEC on the type and acceptability of testing equipment required. Methods for verifying adequate quality assurances and for routine calibration of equipment shall be provided. Chemical and physical guidelines as specified in the latest version of the *Guidelines for Canadian Drinking Water Quality* (*GCDWQ*) shall be considered as objectives, which are applicable to the Province of Newfoundland and Labrador. The *GCDWQ* note that the maximum acceptable concentration (MAC) can be achieved by available water treatment methods at reasonable cost and it must also be reliably measurable by available analytical methods. If it is determined that water quality criteria are exceeded, priority should be given to meeting the *GCDWQ* objectives taking into account costs, the degree of exceedance and local factors. Accredited testing laboratories shall be used for spatial chemical and physical analysis of both raw and treated water as may be conducted or as directed by the DOEC. Any in-house testing shall be in accordance with Standard Methods for the examination of water and wastewater, or any approved alternative methods.

Sufficient bench space, adequate ventilation and lighting, storage room, laboratory sink, auxiliary facilities, and any other testing equipment as required by the DOEC shall be provided. Air conditioning may be necessary.

3.1.8. Monitoring Equipment

Water treatment plants shall be provided with continuous monitoring equipment such as Supervisory Control and Data Acquisition (SCADA) equipment to monitor water being discharged to the distribution system as follows:

1. Plants treating surface water and plants using lime for softening should have the capability to monitor the parameters to evaluate adequate CT disinfection including turbidity, pH, temperature, and free chlorine residual; and

2. Plants treating groundwater using iron removal and/or iron exchange softening should have the capability to monitor and record free chlorine residual.

3.1.9. Sample Taps

Sample taps shall be provided so that water samples can be obtained from each water source and from appropriate locations in each unit operation of treatment. Taps shall be consistent with sampling needs and shall not be of the petcock type. Taps used for obtaining samples for bacteriological analysis shall be of the smooth-nosed type without interior or exterior threads, shall not be of the mixing type, and shall not have a screen, aerator, or other such appurtenance.

3.1.10. Facility Water Supply

The facility water supply service line and the plant finished water sample tap shall be supplied from a source of finished water at a point where all chemicals have been thoroughly mixed, and the required disinfectant contact time has been achieved. There shall be no cross-connections between the facility water supply service line and any piping, troughs, tanks, or other treatment units containing wastewater, treatment chemicals, raw, or partially treated water.

3.1.11. Wall Castings

Extra wall castings should be built into the structure to facilitate future uses whenever pipe passes through walls of concrete structures.

3.1.12. Meters

All water supplies and treatment facilities shall have an acceptable means of metering the finished water. Electronic data loggers that record and store data as well as totalled flow data are recommended.

3.1.13. Piping Colour Code

To facilitate identification of piping in plants and pumping stations, it is recommended that the colour schemes outlined in Tables 3.1, 3.2, 3.3, and 3.4 be utilized:

3.1.13.1. Water Lines

Table 3.1
Piping Colour Code for Water Lines

Line Contents	Colour
Raw	Olive Green
Settled or Clarified	Aqua
Finished or Portable	Dark Blue

3.1.13.2. Chemical Lines

Table 3.2
Piping Colour Code for Chemical Lines

Line Contents	Colour
Alum or Primary Coagulant	Orange
Ammonia	White
Carbon Slurry	Black
Chlorine (Gas and Solution)	Yellow
Fluoride	Light Blue With Red Band
Lime Slurry	Light Green
Ozone	Yellow with Orange Band
Phosphate Compounds	Light Green with Red Band
Polymers or Coagulant Aids	Orange with Green Band
Potassium Permanganate	Violet
Soda Ash	Light Green with Orange Band
Sulphuric Acid	Yellow with Red Band
Sulphur Dioxide	Light Green With Yellow Band

3.1.13.3. Waste Lines

Table 3.3
Piping Colour Code for Waste Lines

Line Contents	Colour
Backwash Waste	Light Brown
Sludge	Dark Brown
Sewer (Sanitary or Other)	Dark Gray

3.1.13.4. Other

Table 3.4
Piping Colour Code for Other Types of Lines

Line Contents	Colour
Compressed Air	Dark Green
Gas	Red
Other Lines	Light Gray

In situations where two colours do not have sufficient contrast to easily differentiate between them, a 150 mm band of contrasting colour should be painted on one of the pipes at approximately 1-metre intervals. The name of the liquid or gas should also be painted on the pipe. In some cases it may be advantageous to paint arrows indicating the direction of flow.

3.1.14. Disinfection

All wells, pipes, tanks, and equipment, which can convey or store potable water, shall be disinfected in accordance with the latest applicable AWWA standards. Plans or specifications shall outline the procedure, and include the disinfectant dosage, contact time, and method of testing the results of the procedure.

3.1.15. Operation and Maintenance Manuals and Parts Lists

An operation and maintenance manual including a parts list and parts order form, operator safety procedures and an operational trouble-shooting section shall be supplied to the water works as part of any proprietary unit installed in the facility.

3.1.16. Operator Instruction

Provisions shall be made for operator instruction at the start-up of a plant or pumping station and follow-up review and additional instruction after a period of 3 months of operation.

3.1.17. Other Considerations

Consideration must be given to the design requirements of other federal, provincial, and local regulatory agencies for items such as safety requirements, special designs for the physically challenged, national building code including national plumbing and electrical codes, construction in flood plains, etc.

3.2 Source Development

In selecting the source of water to be developed, the design engineer must prove to the satisfaction of the DOEC that an adequate quantity of water will be available, and that the water which is to be delivered to the consumers will meet the current requirements of the DOEC with respect to microbiological, physical, chemical and radiological qualities. Each water supply should take its raw water from the best available source, which is economically reasonable and technically possible.

3.2.1. Environmental Assessment Process for Source Development

For new public water supplies, and environmental assessment review may be required for projects where:

1. The supply or works is on a schedule salmon river under the *Fisheries Act*;

2. Inter-basin or intra-basin transfers occur; or

3. Construction of roads. Electric power transmission lines or trunk pipelines for the transmission of water will be located more than 500 m from an existing right of way.

The Environmental Assessment Division of the DOEC should be contacted for additional information or guidance.

3.2.2. Control of Organic Contamination for Public Water Supplies

Although standards and advisories for organics are being developed, there have been numerous cases of organic contamination of public water supply sources. In all cases, public exposure to organic contamination must be minimized. There is insufficient experience to establish design standards, which would apply to all situations. Controlling organic contamination is an area of design that requires pilot studies and early consultation with the DOEC. Where treatment is proposed, best available technology shall be provided to reduce organic contaminants to the lowest practical levels. Operations and monitoring must also be considered in selecting the best alternative. The following alternatives may be applicable:

1. Alternate Source Development;

2. Existing Treatment Modifications;

3. Air Stripping For Volatile Organics (refer to Section 3.3.4.5); and

4. Granular Activated Carbon - Consideration should be given to:
   a. Using contact units rather than replacing portion of existing filter media;

   b. Series and parallel flow piping configurations to minimize the effect of break through without reliance on continuous monitoring;

   c. Providing at least two units. Where only two units are provided, each shall be capable of meeting the plant design capacity (normally the projected maximum daily demand) at the approved rate. Where more than two units are provided, the contactors shall be capable of meeting the design capacity at the approved rate with one or more (as determined in conjunction with the DOEC) units removed from service;

   d. Virgin carbon is the preferred media. Although reactivated carbon may eventually present an economic advantage at large water treatment plants, such an alternative may be pursued only with the preliminary endorsement of the DOEC. Regenerated carbon using only carbon previously used for potable water treatment can be used for this purpose. Transportation and regeneration facilities must not have been used for carbon put to any other use; and

e. Acceptable means of spent carbon disposal.

Except for temporary, emergency treatment conditions, particular attention should be given to developing a Technical Report, which, in addition to the normal determinations, includes the following:

1. For organics contaminants found in surface water sources:
   a. Type of organic chemicals, sources, concentration, frequency of occurrence, water pollution abatement schedule, etc.;

   b. Possible existing treatment plant modifications to lower organic chemical levels. Results of bench, pilot or full scale testing demonstrating treatment alternative, effectiveness and costs; and

   c. A determination of the quality and/or operational parameters which serve as the best measurement of treatment performance, and a corresponding monitoring and process control program.

2. For organic contamination found in groundwater sources:
   a. Types of organic chemicals, sources, concentration, estimate of residence time with the aquifer, plume delineation, flow characteristics, water pollution abatement schedule, etc.;

   b. Results of bench or pilot studies demonstrating treatment alternative, effectiveness, and costs;

   c. A determination of the quality and/or operational parameters which serve as the best measure of treatment performance, and a corresponding monitoring and process control program; and

   d. Development and implementation of a wellhead protection plan.

The collection of this type of data is often complicated and lengthy. Permanent engineering solutions will take a significant time to develop. The cost of organic analyses and the availability of acceptable laboratories may further complicate both pilot work and actual operation.

Alternative source development or purchase of water from nearby unaffected systems may be a more expedient solution for contaminated groundwater sources.

3.2.3. Surface Water

A surface water source includes all tributary streams and drainage basins, natural lakes and artificial reservoirs or impoundments above the point of water supply intake. A source water protection plan for continued protection of the watershed from potential sources of contamination shall be provided as determined by the DOEC.

3.2.3.1. Surface Water Quantity Assessment

A surface water quantity assessment should include a review of the available yield of the water supply. The surface water quantity assessment should demonstrate that:

1. Where possible, a minimum drought return period of one in fifty years has been used for calculating the safe yield;

2. A minimum drought duration of 30 days has been used;

3. The yield is adequate to provide ample water for other legal users of the source including any required fish flows;

4. The yield is adequate to meet the maximum current and future water demand including any required fish flows without significantly affecting the watercourse habitat downstream of the intake; and

5. Only live storage has been used in the yield calculations.

Where site-specific stream flow data is available, yield can be estimated by generated mass flow curves. The stream flow data should also be used to estimate the minimum perennial yield on record and to estimate a drought return period for that year.

Where site-specific stream flow data is not available but precipitation data is available a stream flow record may be simulated to generate mass flow curves. In doing so the runoff characteristics should adequately reflect the hydrologic and topographic characteristics of the watershed.

Where both site-specific stream flow and precipitation data exists both methods should be used and compared. The more conservative yield should be adopted.

For sites where neither site-specific stream flow data nor precipitation data exists, a variety of methods should be used to assess available yield of the water supply, and include the following:

1. The WRMD publication “A Guide to Storage –Yield analysis at Ungauged River Sites” and its accompanying spreadsheet can be used to provide a preliminary estimate of storage requirements for a desired yield; and

2. The WRMD publication “Estimation of Low Flows for the Island of Newfoundland: A User’s Guide” and its accompanying spreadsheet can be used to provide an estimate of low flows.

3.2.3.2. Quality

A sanitary survey and study shall be made of the factors, both natural and man made, which will affect quality. Such survey and study shall include, but not be limited to:

1. Determining possible future uses of impoundments or reservoirs;
2. Determining degree of control of watershed by owner;
3. Assessing degree of hazard to the supply by accidental spillage of materials that may be toxic, harmful or detrimental to treatment processes;
4. Assessing all waste discharges (point source and non-point sources) and activities that could impact the water supply. The location of each waste discharge shall be shown on a scale map;
5. Obtaining samples over a sufficient period of time to assess the microbiological, physical, chemical and radiological characteristics of the water;
6. Assessing the capability of the proposed treatment process to reduce contaminants to conform to the latest version of the *GCDWQ*; and
7. Consideration of currents, wind and ice conditions, and the effect of confluencing streams.

3.2.3.3. Minimum Treatment

1. The design of the water treatment plant must consider the worst conditions that may exist during the life of the facility.
2. The minimum treatment required shall be determined by the DOEC.
3. Filtration preceded by appropriate pretreatment shall be provided for all surface waters. The DOEC, on a case-by-case basis, may approve exemptions.

3.2.3.4. Design of Intake Structures

Design of intake structures shall provide for:

1. Withdrawal of water from more than one level if quality varies with depth;
2. The conduit and the intake structure should be designed so that the intake ports to the pumps do not draw air. The intake pipe should be laid on a continually rising or falling grade to avoid accumulation of air or gas;
3. Separate facilities for release of less desirable water held in storage;
4. Where frazil ice may be a problem, holding the velocity of flow into the intake structure to a minimum, generally not to exceed 150 mm per second;

5. Intake should have adequate protection against clogging by sediment, debris or ice, flotation and wind and wave pressure;

6. Occasional cleaning of the intake line;

7. Adequate protection against rupture by dragging anchors, ice, etc.;

8. Ports located above the bottom of the stream, lake or impoundment, but at sufficient depth to be kept submerged at low water levels;

9. Where shore wells are not provided, a diversion device capable of keeping large quantities of fish or debris from entering an intake structure;

10. Where necessary, provisions shall be made in the intake structure, in consultation with the DOEC, to control the influx of zebra mussels or other aquatic nuisances.

11. When buried surface water collectors or intake galleries are used, sufficient intake opening area must be provided to minimize inlet headloss. Particular attention should be given to the selection of backfill material in relation to the collector pipe slot size and gradation of the native material over the collector system.

3.2.3.5. Wet Wells

Wet wells shall:

1. Have motors and electrical controls located above grade, and protected from flooding or as may be required by the DOEC;

2. Be accessible;

3. Be designed against floatation;

4. Be equipped with removable or traveling screens before the pipe suction well;

5. Provide for introduction of chlorine or other chemicals in the raw water transmission main, if necessary for quality control;

6. Have intake valves and provisions for backflushing or cleaning by a mechanical device and testing for leaks, where practical; and

7. Have provisions for withstanding surges where necessary.

3.2.3.6. Upground Reservoir

An upground reservoir is a facility into which water is pumped during periods of good quality and high stream flow for future release to treatment facilities. Upground reservoirs shall be constructed to assure that:

1. Water quality is protected by controlling runoff into the reservoir;

2. Dikes are structurally sound and protected against wave action and erosion;

3. Intake structures and devices meet requirements of Section 3.2.3.4;

4. Point of influent flow is separated from the point of withdrawal; and

5. Separate pipes are provided for influent to and effluent from the reservoir.

3.2.3.7. Impoundments and Reservoirs

3.2.3.7.1. Site Preparation

Site preparation shall provide, where applicable:

1. Removal of brush and trees to high water elevation;

2. Protection from floods during construction; and

3. Abandonment of all wells, which will be inundated, in accordance with requirements of the DOEC.

3.2.3.7.2. Construction

Construction may require:

1. Approval from the appropriate regulatory agencies of the safety features for stability and spillway design; and

2. A permit from the DOEC for controlling stream flow or installing a structure on the bed of a stream or waterway.

3.2.4. Groundwater

A groundwater source includes all water obtained from dug, drilled, bored or driven wells, and infiltration lines.

3.2.4.1. Quantity

1. The total developed groundwater source capacity should equal or exceed the design maximum day demand and equal or exceed the design average day demand with the largest producing well out of service.

2. A minimum of two groundwater wells shall be provided.

3. The pump is to be set at the predetermined drawdown level.

3.2.4.2. Auxiliary Power

1. When power failure would result in cessation of minimum essential service, sufficient power should be provided to meet average day demand through:
   a. Connection to at least two independent public sources; or

   b. Portable or in-place auxiliary power.

2. When automatic pre-lubrication of pump bearings is necessary and an auxiliary power supply is provided, the pre-lubrication line should be provided with a valved bypass around the automatic control, or the automatic control shall be wired to the emergency power source.

3.2.4.3. Quality

3.2.4.3.1. Microbiological Quality

After disinfection of each new, modified or reconstructed groundwater source, one or more water samples shall be submitted to a laboratory approved by the DOEC. Microbiological, physical and chemical analysis shall be completed with satisfactory results reported to the DOEC prior to the well being placed into service.

3.2.4.3.2. Physical and Chemical Quality

1. Every new, modified or reconditioned groundwater source shall be examined for applicable physical and chemical characteristics by tests of a representative sample in a laboratory satisfactory to the DOEC, with the results reported to the DOEC.

2. Samples shall be collected at the conclusion of the test pumping procedure and examined as soon as practical.

3. Field determinations of physical and chemical constituents or special sampling procedures may be required by the DOEC.

3.2.4.4. Protection Management

3.2.4.4.1. Well Location

The DOEC shall be consulted prior to design and construction regarding a proposed well location as it relates to required separation between existing and potential sources of contamination and groundwater development. The well location should be selected to minimize the impact on other wells and other water resources.

3.2.4.4.2. Continued Sanitary Protection

Continued sanitary protection of the well site from potential sources of contamination shall be provided either through ownership, zoning, easements, leasing or other means acceptable to the DOEC, fencing of the site may be required by the DOEC.

3.2.4.4.3. Wellhead Protection

A wellhead protection plan for continued protection of the wellhead from potential sources of contamination shall be provided, as determined by the DOEC.

3.2.4.5. Testing and Records

3.2.4.5.1. Yield and Drawdown Tests

Yield and drawdown tests shall:

1. Be performed on every production well after construction or subsequent treatment and prior to placement of the permanent pump;

2. Have the test methods clearly indicated in the project specifications;

3. Have a test pump capacity, at maximum anticipated drawdown, at least 1.5 times the quantity anticipated;

4. Provide for continuous pumping for at least 24 hours at the design pumping rate or until stabilized drawdown has continued for at least 6 hours when test pumped at 1.5 times the design pumping rate, and

5. Provide the following data:
   a) Test pump capacity-head characteristics;
   b) Static water level;
   c) Depth of test pump setting;
   d) Time of starting and ending each test cycle; and
   e) The zone of influence for the well or wells.

3.2.4.5.2. Plumbness and Alignment Requirements

1. Every well shall be tested for plumbness and alignment in accordance with AWWA standards.

2. The test method and allowable tolerance shall be clearly stated in the specifications.

3. If the well fails to meet these requirements, the engineer may accept it if it does not interfere with the installation or operation of the pump or uniform placement of grout.

3.2.4.5.3. Geological Data

Geological data shall:

1. Be determined from samples collected at 1.5 m intervals and at each pronounced change in formation;

2. Be recorded, and samples submitted to the DOEC; and

3. Be supplemented with information on accurate records of drillhole diameters and depths, assembled order of size and length of casing and liners, grouting depths, formations penetrated, water levels, and location of any blast charges.

3.2.4.6. General Well Construction

3.2.4.6.1. Drilling Fluids and Additives
Drilling fluids and additives shall:

1. Not impart any toxic substances to the water or promote bacterial contamination; and

2. Be acceptable to the DOEC.

3.2.4.6.2. Minimum Protected Depths
Minimum protected depths of drilled wells shall provide watertight construction to such depth as may be required by the DOEC, to

1. Exclude contamination; and

2. Seal off formations that are, or may be, contaminated or yield undesirable water.

3.2.4.6.3. Temporary Steel Casing
Temporary steel casing used for construction shall be capable of withstanding the structural load imposed during its installation and removal.

3.2.4.6.4. Permanent Steel Casing Pipe
Permanent steel casing pipe shall:
1. Be new single steel casing pipe meeting AWWA Standard A-100, ASTM or API specifications for water well construction;

2. Have minimum weights and thickness indicated in Table 3.5; Page 3-15

3. Have additional thickness and weight if minimum thickness is not considered sufficient to assure reasonable life of a well;

4. Be capable of withstanding forces to which it is subjected;

5. Be equipped with a drive shoe when driven; and

6. Have full circumferential welds or threaded coupling joints.

Table 3.5
Permanent Steel Casing Pipe Minimum Weight and Thickness

Steel Pipe					
Size (mm)	Diameter (mm)		Thickness (mm)	Weight per Foot (kg)	
	External	Internal		Plain Ends (calculated)	With Threads and Couplings (nominal)
150 id	168.275	154.051	7.112	8.605	8.700
200	219.075	202.717	8.179	12.950	13.313
250	255.905	254.508	9.271	18.361	18.983
300	323.850	304.800	9.525	22.480	23.201
350 od.	355.600	336.550	9.525	24.753	25.855
400	406.400	387.350	9.525	28.386	
450	457.200	438.150	9.525	32.019	
500	508.000	488.950	9.525	35.652	
550	558.800	533.400	12.700	52.077	
600	609.600	584.200	12.700	56.921	
650	660.400	635.000	12.700	61.766	
700	711.200	685.800	12.700	66.610	
750	762.000	736.600	12.700	71.454	
800	812.800	787.400	12.700	76.299	
850	863.600	838.200	12.700	81.143	
900	914.400	889.000	12.700	85.998	

3.2.4.6.5. Nonferrous Casing Materials

1. Approval of the use of any nonferrous material as well casing shall be subject to special determination by the DOEC prior to submission of plans and specifications.

2. Nonferrous material proposed as a well casing must be resistant to the corrosiveness of the water and to the stresses to which it will be subjected during installation, grouting and operation.

3.2.4.6.6. Packers

Packers shall be of a material that will not impart taste, odour, toxic substances or bacterial contamination to the well water. Lead packers shall not be used.

3.2.4.6.7. Screens

Screens shall:

1. Be constructed of materials resistant to damage by chemical action of groundwater or cleaning operations;

2. Have a size of openings based on sieve analysis of formation and/or gravel pack materials;

3. Have sufficient length and diameter to provide adequate specific capacity and low aperture entrance velocity. Usually the entrance velocity should not exceed 30 mm/s;

4. Be installed so that the pumping water level remains above the screen under all operation conditions;

5. Where applicable, be designed and installed to permit removal or replacement without adversely affecting water-tight construction of the well; and

6. Be provided with a bottom plate or washdown bottom fitting of the same material as the screen.

3.2.4.6.8. Grouting Requirements

All permanent well casing, except driven Schedule 40 steel casing with the approval of the DOEC, shall be surrounded by a minimum of 37 mm of grout to the depth required by the DOEC. All temporary construction casing shall be removed. Where removal is not possible or practical, the casing shall be withdrawn at least 1.5 m to insure grout contact with the native formation.

1. **Neat cement grout**
    a. Cement conforming to ASTM standard C150 and water, with not more than 23 liters of water per sack of cement, must be used for 37 mm openings.

    b. Additives may be used to increase fluidity subject to approval by the DOEC.

2. **Concrete grout**
    a. Equal parts of cement conforming to AWWA A100 Section 7, and sand, with not more 23 liters of water per sack of cement may be used for openings larger than 37 mm.

    b. Where an annular opening larger than 100 mm is available, gravel not larger than 12 mm in size may be added.

3. **Clay seal**
    a. Where an annular opening greater than 150 mm is available a clay seal of clean local clay mixed with at least 10 per cent swelling bentonite may be used when approved by the DOEC.

4. **Application**
    a. Sufficient annular opening shall be provided to permit a minimum of 37 mm of grout around permanent casings, including couplings.

    b. Prior to grouting through creviced or fractured formations, bentonite or similar materials may be added to the annular opening, in the manner indicated for grouting.

c. When the annular opening is less than 100 mm, grout shall be installed under pressure by means of a grout pump from the bottom of the annular opening upward in one continuous operation until the annular opening is filled.

d. When the annular opening is 100 mm or more and less than 30 m in depth, and concrete grout is used, it may be placed by gravity through a grout pipe installed to the bottom of the annular opening in one continuous operation until the annular is filled.

e. When the annular opening exceeds 150 mm, is less than 30 m in depth, and a clay seal is used, it may be placed by gravity.

f. After cement grouting is applied, work on the well shall be discontinued until the cement grout has properly set.

5. Guides

a. The casing must be provided with sufficient guides welded to the casing to permit unobstructed flow and uniform thickness of grout.

3.2.4.6.9. Upper Terminal Well Construction

1. Permanent casing for all groundwater sources shall project at least 0.3 m above the pumphouse floor or concrete apron surface and at least 0.45 m above final ground surface.

2. Where a well house is constructed, the floor surface shall be at least 150 mm above the final ground elevation.

3. Sites subject to flooding shall be provided with an earth mound to raise the pumphouse floor to an elevation at least 0.6 m above the highest known flood elevation, or other suitable protection as determined by the DOEC.

4. The top of the well casing at sites subject to flooding shall terminate at lest 0.9 m above the 100-year flood level or the highest known flood elevation, whichever is higher, or as the DOEC directs.

3.2.4.6.10. Development

1. Every well shall be developed to remove the native silts and clays, drilling mud or finer fraction of the gravel pack.

2. Development should continue until the maximum specific capacity is obtained from the completed well.

3. Where chemical conditioning is required, the specifications shall include provisions for the method, equipment, chemicals, testing for residual chemicals, and disposal of waste and inhibitors.

4. Where blasting procedures may be used, the specifications shall include the provisions for blasting and cleaning. Special attention shall be given to assure that blasting does not damage the grouting and casing.

3.2.4.6.11. Disinfection of New, Modified or Reconditioned Groundwater Sources

Disinfection of every new, modified or reconditioned groundwater source shall be provided:

1. After completion of work, if a substantial period elapses prior to test pumping or placement of permanent pumping equipment; and

2. After placement of permanent pumping equipment.

3.2.4.6.12. Capping Requirements

1. A welded metal plate or a threaded cap is the preferred method for capping a well.

2. At all times during the progress of work, the contractor shall provide protection to prevent tampering with the well or entrance of foreign materials.

3.2.4.6.13. Well Abandonment

1. Test wells and groundwater sources, which are not in use, shall be sealed by such methods as necessary to restore the controlling geological conditions, which existed prior to construction, or as directed by the DOEC in the *Guidelines for Sealing Groundwater Wells*.

2. Wells to be abandoned shall:
   a. Be sealed to prevent undesirable exchange of water from aquifer to another;

   b. Preferably be filled with neat cement grout;

   c. Have fill materials other than cement grout or concrete, disinfected and free of foreign materials; and

   d. When filled with cement grout or concrete, these materials shall be applied to the well hole through a pipe, tremie, or bailer.

3.2.4.7. Aquifer Types and Construction Methods – Special Conditions

3.2.4.7.1. Sand or Gravel Wells

1. If clay or hard pan is encountered above the water bearing formation, the permanent casing and grout shall extend through such materials.

2. If a sand or gravel aquifer is overlaid only by permeable soils the permanent casing and grout shall extend to at least 6 m below original and final ground elevation, whichever is lower.

3. If a temporary outer casing is used, it shall be completely withdrawn as grout is applied.

3.2.4.7.2. Gravel Pack Wells

1. Gravel pack shall be well rounded particles, 95% siliceous material, that are smooth and uniform, free of foreign material, properly sized, washed and then disinfected immediately prior to or during placement.

2. Gravel pack shall be placed in one uniform continuous operation.

3. Gravel refill pipes, when used, shall be Schedule 40 steel pipe incorporated within the pump foundation and terminated with screwed or welded caps at least 0.3 m above the pump house floor or concrete apron.

4. Gravel refill pipes located in the grouted annular opening shall be surrounded by a minimum of 37 mm of grout.

5. Protection from leakage of grout into the gravel pack or screen shall be provided.

6. Permanent inner and outer casings shall meet requirements of Section 3.2.4.6.4.

7. Minimum casing and grouted depth shall be acceptable to the DOEC.

3.2.4.7.3. Radial Water Collector

1. Locations of all caisson construction joints and porthole assemblies shall be indicated.

2. The caisson wall shall be reinforced to withstand the forces to which it will be subjected.

3. Radial collectors shall be in areas and at depths approved by the DOEC.

4. Provisions shall be made to assure that radial collectors are essentially horizontal.

5. The top of the caisson shall be covered with a watertight floor.

6. All openings in the floor shall be curbed and protected from entrance of foreign material.

7. The pump discharge piping shall not be placed through the caisson walls. In unique situations where this is not feasible, a watertight seal must be obtained at the wall.

3.2.4.7.4. Infiltration Lines

1. Infiltration lines may be considered only where geological conditions preclude the possibility of developing an acceptable drilled well.

2. The area around infiltration lines shall be under the control of the water purveyor for a distance acceptable to, or required by the DOEC.

3. Flow in the lines shall be by gravity to the collecting well.

3.2.4.7.5. Dug Wells

1. Dug wells may be considered as a source of public water supply where site-specific conditions permit.

2. A watertight cover shall be provided.

3. Minimum protective lining and grouted depth shall be at least 3 m below original, or final ground elevation, whichever is lower.

4. Openings shall be curbed and protected from entrance of foreign materials.

5. Pump discharge piping shall not be placed through the well casing or wall.

3.2.4.7.6. Limestone or Sandstone Wells

1. Where the depth of unconsolidated formations is more than 15 m, the permanent casing shall be firmly seated in uncreviced or unbroken rock. Grouting requirements shall be determined by the DOEC.

2. Where the depth of unconsolidated formations is less than 15 m, the depth of casing and grout shall be at least 15 m or as determined by the DOEC.

3.2.4.7.7. Naturally Flowing Wells

1. Flow shall be controlled.

2. Permanent casing and grout shall be provided.

3. If erosion of the confining bed appears likely, special protective construction may be required by the DOEC.

3.2.4.8. Well Pumps, Discharge Piping and Appurtenances

3.2.4.8.1. Line Shaft Pumps

Wells equipped with line shaft pumps shall:

1. Have the casing firmly connected to the pump structure or have the casing inserted into a recess extending at least 12 mm into the pump base;

2. Have the pump foundation and base designed to prevent water from coming into contact with the joint; and

3. Avoid the use of oil lubrication at pump settings less than 122 m.

3.2.4.8.2. Submersible Pumps

Where a submersible pump is used:

1. The top of the casing shall be effectively sealed against the entrance of water under all conditions of vibration or movement of conductors or cables; and

2. The electrical cable shall be firmly attached to the riser pipe at 6 m intervals or less.

3.2.4.8.3. Discharge Piping

The discharge piping shall:

1. Be designed so that the friction loss will be low;

2. Have control valves and appurtenances located above the pump house floor when an above-ground discharge is provided;

3. Be protected against the entrance of contamination;

4. Be equipped with a check valve, shutoff valve, a pressure gauge, a means of measuring flow, and a smooth nosed sampling tap located at a point where positive pressure is maintained;

5. Where applicable, be equipped with an air release-vacuum relief valve located upstream from the check valve, with exhaust/relief piping terminating in a down-turned position at least 0.71 m above the floor and covered with a 24-mesh corrosion resistant screen;

6. Be valved to permit test pumping and control of each well;

7. Have all exposed piping, valves and appurtenances protected against physical damage and freezing;

8. Be properly anchored to prevent movement; and

9. Be protected against surge or water hammer.

The discharge piping should be provided with a means of pumping to waste, but shall not be directly connected to a sewer.

3.2.4.8.4. Pitless Well Units

1. The DOEC must be contacted for approval of specific applications of pitless units.

2. Pitless units shall:

    a. Be shop-fabricated from the point of connection with the well casing to the unit cap or cover;

    b. Be threaded or welded to the well casing;

    c. Be of watertight construction throughout;

    d. Be of materials and weight at least equivalent and compatible to the casing;

    e. Have field connection to the lateral discharge from the pitless unit of threaded, flanged or mechanical joint connection; and

f. Terminate at least 0.45 m above final ground elevation or 0.9 m above the 100-year flood level or the highest known flood elevation, whichever is higher, or as directed by the DOEC.

3. The design of the pitless unit shall make provisions for:

   a. Access to disinfect the well;

   b. A properly constructed casing vent meeting the requirements of Section 3.2.4.8.5;

   c. Facilities to measure water levels in the well, according to Section 3.2.4.8.6;

   d. A cover at the upper terminal of the well that will prevent the entrance of contamination;

   e. A contamination-proof entrance connection for electrical cable,

   f. An inside diameter as great as that of the well casing, up to and including casing diameters of 0.3 m, to facilitate work and repair on the well, pump, or well screen; and

   g. At least one check valve within the well casing or in compliance with requirements of the DOEC.

4. If the connection to the casing is by field weld, the shop-assembled unit must be designed specifically for field welding to the casing. The only field welding permitted will be that needed to connect a pitless unit to the casing.

3.2.4.8.5. Casing Vent

Provisions shall be made for venting the well casing to atmosphere. The vent shall terminate in a down turned position, at or above the top of the casing or pitless unit in a minimum 37 mm diameter opening covered with a 24 mesh, corrosion resistant screen. The pipe connecting the casing to the vent shall be of adequate size to provide rapid venting of the casing.

3.2.4.8.6. Water Level Measurement

1. Provision shall be made for periodic measurement of water levels in the completed well.

2. Where pneumatic water level measuring equipment is used it shall be made using corrosion resistant materials attached firmly to the top pipe or pump column and in such a manner as to prevent entrance of foreign materials.

3.2.4.8.7. Observation Wells

Observation wells shall be:

1. Constructed in accordance with the requirements for permanent wells if they are to remain in service after completion of a water supply well; and

2. Protected at the upper terminal to preclude entrance of foreign materials.

3.2.5. Langelier Index

The Langelier Index (LI) is an approximate measure of the degree of saturation of calcium carbonate in water. It is calculated using the pH, alkalinity, hardness, total dissolved solids, and water temperature. It is dependent on temperature and will vary with water temperature.

1. If the LI is negative – The water is under saturated with calcium carbonate and will tend to be corrosive in the distribution system.

2. If the LI is positive – The water is over saturated with calcium carbonate and will tend to deposit calcium carbonate forming scales in the distribution system.

3. If the LI is close to zero – The water is just saturated with calcium carbonate and will neither be strongly corrosive nor scale forming.

The LI is one of several tools used by the water operator for stabilizing water to control both corrosion and the deposition of scale. To ensure the long life of the distribution system it is best to have a LI close to one.

3.2.6. pH Adjustment

When first determining the amount of treatment required for a new water system or the upgrade of an existing water system, the pH of the raw water must be measured along with the approximate degree of saturation of calcium carbonate in the water using the Langelier Saturation Index. The Langelier Saturation Index will determine the potential corrosiveness of the water to be treated.

The pH of the raw water and the potential corrosiveness of the water are extremely important when the only form of treatment to be provided is disinfection using chlorine.

Most surface water in the province has very little alkalinity while some groundwater have excessive amounts of hardness. Based upon past experience, it has been shown that low alkalinity water, disinfected with gas chlorine, has a tendency to significantly lower the pH, which in turn causes excessive corrosiveness in the distribution system. In groundwater with a pH higher than 8, the disinfecting capability of chlorine, at this pH level, is compromised. The use of sodium hypochlorite in groundwater may raise the pH of the water even higher, which will affect disinfection. In such cases, the pH of water may have to be lowered.

3.2.7. Source Protection

Owners of water supply systems must apply to the DOEC as per Section 39 of the *Water Resources Act* for source and wellhead protection.

3.3. Water Treatment

The design of treatment processes and devices shall depend on the evaluation of the nature and quality of the particular water to be treated, seasonal variations, the desired quality of the finished water, and the mode of operation planned.

Water treatment facilities should be designed such that major process equipment and facilities are capable of supplying the maximum day demand for the 20 to 25 year projected design flows, plus an additional amount that will be sufficient to accommodate plant losses. Maximum Day demand is the maximum amount of water supplied to the system on any given day within a calendar year. Peak flows are the short-term flows expected to be experienced by a particular component of the system and will govern the sizing of many system components.

Minor process equipment such as piping, valves and chemical feed systems should be designed to accommodate future design flow, within the life expectancy of the components. Water treatment facilities should be designed to facilitate future expansion, if necessary. In each case, the designer may consider modularity and expandability as an option to the provision of surplus capacity.

3.3.1. Clarification

Plants designed for processing surface water shall:

1. Provide a minimum of two units each for rapid mix, flocculation and sedimentation;

2. Permit operation of the units either in series or parallel where softening is performed, and should permit series or parallel operation where plain clarification is performed;

3. Be constructed to permit units to be taken out of service without disrupting operation, and with drains or pumps sized to allow dewatering in a reasonable period of time;

4. Provide multiple-stage treatment facilities when required by the DOEC;

5. Be started manually following shutdown; and

6. Minimize hydraulic head losses between units to allow future changes in processes without the need for re-pumping.

3.3.1.1. Presedimentation

Waters containing high turbidity may require pre-treatment, usually sedimentation either with or without the addition of coagulation chemicals.

1. **Basin design –** presedimentation basins should have hopper bottoms or be equipped with continuous mechanical sludge removal apparatus, and provide arrangements for dewatering.

2. **Inlet –** incoming water shall be dispersed across the full width of the line of travel as quickly as possible: short-circuiting must be prevented.

3. **Bypass –** provisions for bypassing presedimentation basins shall be included.

4. **Detention time –** 3 hours detention is the minimum period recommended; greater detention may be required.

3.3.1.2. Rapid Mix

Rapid mix means the rapid dispersion of chemicals throughout the water to be treated, usually by violent agitation. The engineer shall submit the design basis for the velocity gradient (G value) selected, considering the chemicals to be added and water quality parameters.

1. **Equipment –** basins should be equipped with mechanical mixing devices. Static mixing maybe considered if treatment flow is not variable and can be justified by the design engineer.

2. **Mixing –** detention period should be not more than 30 seconds.

3. **Location** - the rapid mix and flocculation basin shall be as close together as possible.

3.3.1.3. Flocculation

Flocculation is the agitation of water at low velocities to promote the formation and growth of a settleable floc.

Inlet and outlet design should prevent short-circuiting and destruction of floc. Basin drains should be provided and should be a minimum of 200 mm diameter.

3.3.1.3.1. Detention
The flow-through velocity shall be no less than 0.15 m or greater than 0.45 m/min with a detention time of at least 30 min.

3.3.1.3.2. Equipment
Agitators shall be driven by variable speed drives with the peripheral paddle speed ranging from 0.15 to 0.6 m/s.

3.3.1.3.3. Piping
Flocculation and sedimentation basins should be as close together as possible. The velocity of flocculated water through pipes or conduits to settling basins shall be between 0.15 and 0.6 m/s. Allowances must be made to minimize turbulence at bends and changes in direction.

3.3.1.3.4. Alternate Design
Baffling may be used to provide for flocculation in small plants only after consultation with the DOEC. The design should be such that the velocities and flows noted above would be maintained.

3.3.1.3.5. Superstructure
A structure over the flocculation basins may be required.

3.3.1.4. Sedimentation

Sedimentation shall follow flocculation and the basins should provide quiescent settling for the removal of floc and other suspended solids. Basins may be either rectangular or circular and should have continuous mechanical sludge removal equipment. The detention time for effective clarification is dependent upon a number of factors related to basin design and the nature of the raw water. The following criteria apply to conventional sedimentation units.

3.3.1.4.1. Detention
The minimum detention time should be 4 hours. This may be reduced to 2 hours for lime-soda softening facilities treating only groundwater. Reduced sedimentation time may also be approved when equivalent effective settling is demonstrated, or when overflow rate is not more than 1.2 m/hr.

3.3.1.4.2. Inlet
Inlets shall be designed to distribute the water equally and at uniform velocities. Open ports, submerged ports, and similar entrance arrangements are required. A baffle should be constructed across the basin, close to the inlet end, and should project a sufficient distance below the water surface to dissipate inlet velocities and provide uniform flows across the basin.

3.3.1.4.3. Outlet
Outlet weirs or submerged orifices shall maintain velocities suitable for settling in the basin and to minimize short-circuiting. The use of submerged orifices is recommended in order to provide a volume above the orifices when there are fluctuations in flow. Outlet weirs and submerged orifices shall be designed as follows:

1. The rate of flow over the outlet weirs or through the submerged orifices shall not exceed 250 m^3/day/m of the outlet launder;

2. Submerged orifices should not be located lower than 0.9 m below the flow line; and

3. The entrance velocity through the submerged orifices shall not exceed 0.15 m/s.

3.3.1.4.4. Velocity
The velocity through settling basins should not exceed 0.15 m/s. The basins must be designed to minimize short-circuiting. Fixed and adjustable baffles must be provided, as necessary, to achieve the maximum potential for clarification.

3.3.1.4.5. Overflow
An overflow weir or pipe, which will establish the maximum water level desired on top of the filters, should be installed. The overflow shall discharge by gravity with a free fall at a location where the discharge will be noted.

3.3.1.4.6. Drainage

Basins must be provided with a means of dewatering. Basin bottoms should slope at approximately 8% toward the drain unless mechanical sludge removal equipment is installed. The discharge of drainage from any mixing or settling tank must be approved by the DOEC.

3.3.1.4.7. Superstructure

A superstructure over the sedimentation basins may be required. If there is no mechanical equipment in the basins, or if provisions are included for adequate monitoring under all expected weather conditions, a cover may be provided in lieu of a superstructure. The cover should be provided with manholes, equipped with raised curb and covers, as well as drop light connections, so that observations can be made, at several points, of the efficiency of sedimentation.

3.3.1.4.8. Sludge Collection

Mechanical sludge collection equipment should be provided.

3.3.1.4.9. Flushing Lines

Flushing lines or hydrants shall be provided, and must be equipped with backflow prevention devices acceptable to the DOEC.

3.3.1.4.10. Safety

Permanent ladders or handholds should be provided on the inside walls of the basin above the water level. Guardrails should be included. Compliance with other applicable safety requirements, such as the *Occupational Health and Safety Act* (OHSA), and regulations under the Act, shall be required.

3.3.1.4.11. Sludge Removal

Sludge removal design shall provide that:

1. Sludge pipes shall be not less than 75 mm in diameter and so arranged as to facilitate cleaning;

2. Entrance to sludge withdrawal piping shall prevent clogging;

3. Valves shall be located outside the tank for accessibility; and

4. The operator may observe and sample sludge being withdrawn from the unit.

3.3.1.4.12. Sludge Disposal

Facilities are required, by the DOEC, for the disposal of sludge (See Section 3.3.12.).

3.3.1.5. Solids Contact Unit

Units are generally acceptable for combined softening and clarification where water characteristics, especially temperature, do not fluctuate rapidly, flow rates are uniform, and operation is continuous. Before such units are considered as clarifiers without softening, specific approval of the DOEC shall be obtained. Clarifiers should be designed for the maximum uniform rate and should be adjustable to changes in flow, which are less than the design rate, and for changes in water characteristics. A minimum of two units, are required for surface water treatment.

3.3.1.5.1. Installation of Equipment

Supervision by a representative of the manufacturer shall be provided, with regard to all mechanical equipment, at the time of installation and initial operation.

3.3.1.5.2. Operating Equipment

The following shall be provided for plant operation:

1. A complete outfit of tools and accessories;

2. Necessary laboratory equipment; and

3. Adequate piping with suitable sampling taps located so as to permit the collection of water samples from critical portions of the units.

3.3.1.5.3. Chemical Feed

Chemicals shall be applied at such points and by such means as to insure satisfactory mixing of the chemicals with water.

3.3.1.5.4. Mixing

A rapid mix device or chamber ahead of solids contact units may be required by the DOEC to ensure proper mixing of the chemicals applied. Mixing devices shall be constructed to:

1. Provide good mixing of the raw water with previously formed sludge particles; and

2. Prevent deposition of solids in the mixing zone.

3.3.1.5.5. Flocculation

Flocculation equipment:

1. Shall be adjustable (speed and/or pitch);

2. Must provide for coagulation in separate chamber of baffled zone within the unit; and

3. Should provide the flocculation and mixing period to be not less than 30 minutes.

3.3.1.5.6. Sludge Concentrators

1. The equipment should provide either internal or external concentrators in order to obtain a concentrated sludge with a minimum of wastewater.

2. Large basins should have at least two sumps for collecting sludge located in the central flocculation zone.

3.3.1.5.7. Sludge Removal

Design of sludge removal process shall be as per Section 3.3.1.4.11.

3.3.1.5.8. Cross-connections

1. Blow-off outlets and drains must terminate and discharge at places satisfactory to the DOEC.

2. Cross-connection control must be included for potable water lines used to backflush sludge lines.

3.3.1.5.9. Detention Period
The detention time shall be established on the basis of the raw water characteristics and other local conditions that affect the operation of the unit. Based on design flow rates, the detention time should be:

1. 2 to 4 hours for suspended solids contact clarifiers and softeners treating surface water; and

2. 1 to 2 hours for the suspended solids contact softeners treating only groundwater.

The DOEC may alter detention time requirements.

3.3.1.5.10. Suspended Slurry Concentrate
Softening units should be designed so that continuous slurry concentrates of 1% or more, by weight, can be satisfactorily maintained.

3.3.1.5.11. Water Losses
1. Units shall be provided with suitable controls for sludge withdrawal.

2. Total water losses should not exceed 5% for clarifiers and 3% for softening units.

3. Solids concentration of sludge bled to waste should be 3% by weight for clarifiers and 5% by weight for softeners.

3.3.1.5.12. Weirs or Orifices
The units should be equipped with either overflow weirs or orifices constructed so that water at the surface of the unit does not travel over 3 m horizontally to the collection trough.

1. Weirs shall be adjustable, and at least equivalent in length to the perimeter of the tank.

    a) Weir loading shall not exceed 120 L/min/m of weir length for units used for clarifiers, and 240 L/min/m of weir length for units used for softeners.

2. Where orifices are used, the loading rates per foot of launder rates should be equivalent to weir loadings. Either shall produce uniform rising rates over the entire area of the tank.

3.3.1.5.13. Upflow Rates
Unless supporting data is submitted to the DOEC to justify rates exceeding the following, rates shall not exceed:

1. 2.4 m/hr at the sludge separation line for units used for clarifiers; and

2. 4.2 m/hr at the slurry separation line, for units used for softeners.

3.3.1.6. Tube or Plate Settlers

Proposals for tube settler unit clarification must include pilot plant and/or full-scale demonstration data on water with similar quality prior to the preparation of final plans and specifications satisfactory to the DOEC. Settler units consisting of various shaped tubes or plates, which are installed, in multiple layers and at an angle to the flow may be used for sedimentation, following flocculation. The proposal would form part of the Technical Report (see Section 2.1).

1. **Inlet and Outlet Considerations –** Design to maintain velocities suitable for settling in the basin and to minimize short-circuiting. Plate units shall be designed to minimize misdistribution across the units.

2. **Drainage –** Drain piping from the settler units must be sized to facilitate a quick flush of the settler units and to prevent flooding of other portions of the plant.

3. **Protection from Freezing** – Although most units will be located within a plant, outdoor installations must provide sufficient freeboard above the top of the settlers to prevent freezing in the units. A cover or enclosure is strongly recommended.

4. **Application Rate for Tubes –** A maximum rate of 4.8 m/hr for tube settlers, unless higher rates are successfully shown through pilot plant or in-plant demonstration studies.

5. **Application Rates for Plates** – A maximum plate loading rate of 1.2 m/hr based on 80% of the projected horizontal plate area.

6. **Flushing Lines –** Flushing lines shall be provided to facilitate maintenance and must be properly protected against backflow or back-siphonage.

3.3.2. Filtration

Acceptable filters include:
1. Rapid rate gravity filters;
2. Rapid rate pressure filters;
3. Diatomaceous earth filtration;
4. Slow sand filtration;
5. Direct filtration;
6. Deep bed rapid rate gravity filters;
7. Biologically active filters;
8. Membrane filtration;
9. Reverse Osmosis; and
10. Bag and cartridge filters.

The application of any one type must be supported by water quality data representing a reasonable period of time to characterize variations in water quality. Experimental treatment studies may be required to demonstrate the applicability of the method of filtration proposed.

3.3.2.1. Rapid Rate Gravity Filters

3.3.2.1.1. Pretreatment
The use of rapid rate gravity filters shall require pretreatment.

3.3.2.1.2. Rate of Filtration
The rate of filtration should be determined through consideration of such factors as raw water quality, degree of pretreatment provided, filter media, water quality control parameters, competency of operating personnel, and other factors as required by the DOEC. In any case, the filter rate must be proposed and justified by the design engineer to the satisfaction of the DOEC prior to the preparation of the final plans and specifications.

3.3.2.1.3. Number of Units
At a minimum, 2 units shall be provided. Each shall be capable of meeting the plant design capacity (normally the projected maximum daily demand) at the approved filtration rate. Where more than 2 filter units are provided, the filters shall be capable of meeting the plant design capacity at the approved filtration rate with one filter removed from service. Where declining rate filtration is provided, the variable aspect of filtration rates, and the number of filters must be considered when determining the design capacity for the filters.

3.3.2.1.4. Structural Details and Hydraulics
The filter structure should be designed to provide for:

1. Vertical walls within the filter;

2. No protrusion of the filter walls into the filter media;

3. Cover by a superstructure;

4. Head room to permit inspection and operation;

5. Minimum filter box depth of 2.6 m;

6. Minimum water depth over the surface of the filter media of 0.9 m;

7. Filter effluent pipes in the clear well which should be trapped to prevent air from entering the bottom of the filters;

8. Prevention of floor drainage to the filter with a minimum 100 mm curb around the filters;

9. An overflow to prevent water from rising above the walls of the filters;

10. Water levels in the filters to be below the filter operating floor to prevent sweating of the filter walls;

11. A maximum velocity of 0.6 m/s in influent pipes or conduits;

12. The influent pipes and conduits shall be straight with crosses or clean-out chambers at changes in direction or following lime-soda softening to permit cleaning;

13. Washwater drain capacity to carry maximum flow;

14. Walkways around filters, to be not less than 600 mm wide;

15. Safety handrails or walls around all filter walkways; and

16. Construction to prevent cross-connections and common walls between potable and non-potable water.

3.3.2.1.5. Washwater Troughs

Washwater troughs should be constructed to have:

1. The bottom elevation above the maximum level of expanded media during back-washing;

2. A 50 mm freeboard at the maximum rate of wash;

3. The top edge level and all at the same elevation;

4. Spacing so that each trough serves the same number of square meters of filter area; and

5. Maximum horizontal travel of suspended particles to reach the trough not to exceed 0.9 m.

3.3.2.1.6. Filter Material

The media shall be clean silica sand or other natural or synthetic media, approved by the DOEC, having the following characteristics:

1. Total depth of not less than 0.6 m and generally not more than 0.76 m;

2. Effective size range of the smallest material no greater than 0.45 mm to 0.55 mm;

3. Uniformity coefficient of the smallest material not greater than 1.65,

4. A minimum of 0.3 m of media with an effective size range no greater than 0.45 mm to 0.55 mm and a specific gravity greater than other filtering materials within the filter,

5. Types of Filter Media:

   a. ***Anthracite*** - Clean crushed anthracite, or a combination of anthracite and other media may be considered on the basis of experimental data specific to the project and shall have:

      i) Effective size of 0.45 mm – 0.55 mm with uniformity coefficient not greater than 1.65 when used alone;

ii) Effective size of 0.8 mm – 1.2 mm with a uniformity coefficient not greater than 1.85 when used as a cap; and

iii) Effective size for anthracite used as a single media on potable groundwater for iron and manganese removal only shall be a maximum of 0.8 mm (effective sizes greater than 0.8 mm may be approved based upon onsite pilot plant studies or other demonstration acceptable to the DOEC).

If the anthracite grains are too small, excessive losses will be incurred during the minimum backwash that is required to clean the sand effectively.

b. ***Sand*** **–** Shall have effective size of 0.45 mm to 0.55 mm, and uniformity coefficient of not greater than 1.65.

c. ***Granular activated carbon (GAC)*** **–** Granular activated carbon as a single media may be considered for filtration only after pilot or full scale testing and with prior approval of the DOEC. The design shall include the following:

i) The media must meet the basic specifications for filter media as given in Section 3.3.2.1.6 (1) through (4) except that larger size media may be allowed by the DOEC where full scale tests have demonstrated that treatment goals can be met under all conditions;

ii) There must be provisions for a free chlorine residual and adequate contact time in the water following the filters and prior to distribution (see Sections 4.2.1 and 4.2.3);

iii) There must be means for periodic treatment of filter material for control of bacteria and other growth; and

iv) Provisions must be made for frequent replacement or regeneration.

d. ***Torpedo Sand*** **–** A 75 mm layer of torpedo sand should be used as a supporting media for filter sand where supporting gravel is used, and shall have effective size of 0.8 mm to 2.0 mm, and uniformity coefficient not greater than 1.7.

e. ***Gravel*** **-** Gravel, when used as the supporting media, shall consist of cleaned and washed, hard, durable, rounded silica particles and shall not include flat or elongated particles. The coarsest gravel shall be 64 mm in size when the gravel rests directly on a lateral system, and must extend above the top of the perforated laterals. Not less than four layers of gravel shall be provided in accordance with the size and depth distribution, outlined in Table 3.6, when used with perforated laterals.

f. ***Other Media*** **–** Other media will be considered based on experimental data and operating experience.

Table 3.6
Size and Depth Distribution

Size (mm)	Depth (mm)
64 – 38	125 – 205
38 – 19	76 – 125
19 – 12.5	76 – 125
12.5 – 5	50 – 76
5 – 2.5	50 – 76

Reduction of gravel depth and other size gradations may be considered upon justification to the DOEC for slow sand filtration or when proprietary filter bottoms are specified.

3.3.2.1.7. Filter Bottoms and Strainer Systems

Porous plate bottoms shall not be used where iron or manganese may clog them or with water softened with lime. The design of manifold-type collection systems shall:

1. Minimize loss of pressure in the manifold and laterals;

2. Ensure even distribution of washwater and even rate of filtration over the entire area;

3. Provide the ratio of the area of the final openings of the strainer systems to the area of the area of the filter at about 0.003;

4. Provide the total cross-sectional area of the laterals at about twice the total area of the final openings;

5. Provide the cross-sectional area of the manifold at 1.5 to 2 times the total area of the final openings; and

6. Direct lateral perforations without strainers downward.

3.3.2.1.8. Surface Wash or Subsurface Wash

Surface or subsurface wash facilities are required except for filters used exclusively for iron or manganese removal, and may be accomplished by a system of fixed nozzles or a revolving type apparatus. All devices shall be designed with:

1. Provisions for water pressures of at least 310 kPa,

2. A properly installed vacuum breaker or other approved device to prevent back-siphonage if connected to the treated water system;

3. A rate of flow of 1.36 L/m^{2}s of filter areas (4.9 m/hr) with fixed nozzles or 0.34 L/m^{2}s (1.2 m/hr) with revolving arms, and

4. Air wash based on experimental data and operating experiences.

3.3.2.1.9. Air Scouring

Air scouring can be considered in place of surface wash, and shall be designed as follows:

1. Air flow for air scouring the filter must be 0.9 – 1.5 $m^3/min/m^2$ when the air is introduced in the underdrain; a lower air rate must be used when the air scour distribution system is placed above the underdrains;

2. A method for avoiding excessive loss of the filter media during backwashing must be provided;

3. Air scouring must be followed by a fluidization wash sufficient to re-stratify the media;

4. Air must be free from contamination;

5. Air scour distribution systems should be placed below the media and supporting bed interface; if placed at the interface the air scour nozzles shall be designed to prevent media from clogging the nozzles or entering the air distribution system;

6. Piping for the air distribution system shall not be flexible hose which will collapse when not under air pressure, and shall not be a relatively soft material which may erode at the orifice opening with the passage of air at high velocity;

7. Air delivery piping shall not pass down through the filter media, nor shall there be any arrangement in the filter design which would allow short circuiting between the applied unfiltered water and the filtered water;

8. Consideration should be given to maintenance and replacement of air delivery piping;

9. The backwash water delivery system must be capable of 37 $m/hr/m^2$ of surface area; however, when air scour is provided, the backwash water rate must be variable and should not exceed 20 $m/hr/m^2$ of surface area unless operating experience shows that a higher rate is necessary to remove scoured particles from filter media surfaces;

10. The filter underdrains shall be designed to accommodate air scour piping when the piping is installed in the underdrain, and

11. The provisions of Section 3.3.2.1.11 shall be followed.

3.3.2.1.10. Appurtenances

1. The following shall be provided for every filter:

    a. Influent and effluent sampling taps,

    b. An indicating loss of head gauge,

    c. An indicating rate-of-flow meter. A modified rate controller, which limits the rate of filtration to a maximum rate, may be used. However, equipment that simply maintains a constant water level on the filters is not acceptable, unless the rate of flow unto the filter is properly controlled. A pump or a flow meter in each filter effluent line may be used as the limiting device for the rate of filtration only after consultation with the DOEC, and

d. Where used for surface water, provisions for filtering to waste with appropriate measures for backflow prevention.

2. The following should be provided for all filters:

   a. A continuous or rotating cycle turbidity recording device for surface water treatment plants,

   b. Wall sleeves providing access to the filter interior at several locations for sampling or pressure sensing,

   c. A 25 to 38 mm diameter pressure hose and storage rack at the operating floor for washing filter walls,

   d. Provisions for draining the filter to waste with appropriate measures for backflow prevention, and
   e. Particle monitoring equipment as a means to enhance overall treatment options where used for surface water.

3.3.2.1.11. Backwash

Provisions should be made for washing filters as follows:

1. A minimum rate of 37 m/hr, consistent with water temperatures and specific gravity of the filter media. A rate of 50 m/hr or a rate necessary to provide for a 50% expansion of the filter bed is recommended. A reduced rate of 24 m/hr may be acceptable for full depth anthracite or granular activated carbon filters. Backwashing rates may be reduced to 12 to 18 m/hr for systems using air scour;

2. Filtered water provided at the required rate by washwater tanks, a washwater pump, from the high service main, or a combination of these;

3. Washwater pumps in duplicate unless an alternate means of obtaining washwater is available;

4. Not less than 15 minute wash of one filter at the design rate of wash;

5. A washwater regulator or valve on the main washwater line to obtain the desired rate of filter wash with the washwater valves on the individual filters open wide;

6. A rate-of-flow indicator, preferably with a totalizer, on the main washwater line, located so that it can be easily read by the operator during the washing process;

7. Capability to backwash all filters within a 24-hour period;

8. Design to prevent rapid changes in backwash water flow, and

9. Backwash shall be operator initiated; automated systems shall be operator adjustable.

3.3.2.1.12. Miscellaneous

Roof drains shall not discharge into the filters or basins and conduits preceding the filters.

3.3.2.2. Rapid Rate Pressure Filters

The normal use of these filters is for iron and manganese removal. Pressure filters shall not be used in the filtration of polluted waters or following lime-soda softening. Minimum criteria relative to number, rate of filtration, structural details and hydraulics, filter media, etc., provided for rapid rate gravity filters also apply to pressure filters where appropriate.

3.3.2.2.1. Rate of Filtration

The rate should not exceed 7.2 m/hr except where in-plant testing, as approved by the DOEC, has demonstrated satisfactory results at higher rates.

3.3.2.2.2. Details of Design

The filters shall be designed to provide for:

1. Loss of pressure gauges on the inlet and outlet pipes of each filter;

2. An easily readable meter or flow indicator on each battery of filters. A flow indicator is recommended for each filtering unit;

3. Filtration and backwashing of each filter individually with an arrangement of piping as simple as possible to accomplish these purposes;

4. Minimum sidewall shell height of 1.5 m. A corresponding reduction in sidewall height is acceptable where proprietary bottoms permit reduction of the gravel depth;

5. The top of the washwater collectors to be at least 450 mm above the surface of the media;

6. The underdrain system to efficiently collect the filtered water and to uniformly distribute the backwash water at a rate not less than 37 m/hr;

7. Backwash flow indicators and controls that are easily readable while operating the control valves;

8. An air release valve on the highest points of each filter;

9. An accessible manhole to facilitate inspections and repairs for filters 0.9 m or more in diameter. Sufficient handholds shall be provided for filters less than 0.9 m in diameter. Manholes should be at least 0.6 m in diameter where feasible;

10. Means to observe the wastewater during backwashing, and

11. Construction to prevent cross-connection.

3.3.2.3. Diatomaceous Earth Filtration

The use of these filters may be considered for application to surface water with low turbidity and low bacterial contamination, and may be used for iron removal for groundwater providing the removal is effective and the water is of satisfactory sanitary quality before treatment.

3.3.2.3.1. Conditions of Use

Diatomaceous earth filters are expressly excluded from consideration for the following conditions:

1. Bacteria removal,

2. Colour removal,

3. Turbidity removal, where either the gross quantity of turbidity is high or the turbidity exhibits poor filterability characteristics, and

4. Filtration of water with high algae counts.

3.3.2.3.2. Pilot Plant Study

Installation of a diatomaceous earth filtration system shall be preceded by a pilot plant study on the water to be treated.

1. Conditions of the study such as duration, filter rates, head loss accumulation, slurry feed rates, turbidity removal, bacteria removal, etc., must be approved by the DOEC prior to the study.

2. Satisfactory pilot plant results must be obtained prior to preparation of final construction plans and specifications.

3. The pilot plant study must demonstrate the ability of the system to meet the latest version of the *GCDWQ* at all times.

3.3.2.3.3. Types of Filters

Pressure or vacuum diatomaceous earth filtration units will be considered for approval. However, the vacuum type is preferred for its ability to accommodate a design which permits observation of the filter surfaces to determine proper cleaning, damage to a filter element, and adequate coating over the entire filter area.

3.3.2.3.4. Treated Water Storage

Treated water storage capacity in excess of normal requirements shall be provided to:

1. Allow operation of the filters at a uniform rate during all conditions of system demand at or below the approved filtration rate, and

2. Guarantee continuity of the service during adverse raw water conditions without by-passing the system.

3.3.2.3.5. Number of Units

See Section 3.3.2.1.3.

3.3.2.3.6. Pre-coat

1. **Application** - A uniform pre-coat shall be applied hydraulically to each septum by introducing a slurry to the tank influent line and employing a filter-to-waste or recirculation system.

2. **Quantity** - Diatomaceous earth in the amount of 0.5 kg/m^2, or an amount sufficient to apply 1.6 mm coating, should be used with recirculation. When pre-coating is accomplished with a filter-to-waste system, 0.7 - 1.0 kg/m^2 is recommended.

3.3.2.3.7. Body Feed

A body feed system to apply additional amounts of diatomaceous earth slurry during the filter run is required to avoid short filter runs or excessive head losses.

1. Quantity - Rate of body feed is dependent on raw water quality and characteristics, and must be determined in the pilot plant study.

2. Operation and maintenance can be simplified by providing accessibility to the feed system and slurry lines.

3. Continuous mixing of the body feed slurry is required.

3.3.2.3.8. Filtration

1. **Rate of Filtration** - The recommended normal rate is 2.4 m/hr with a recommended maximum of 3.7 m/hr. The filtration rate shall be controlled by a positive means.

2. **Head Loss –** The head loss shall not exceed 210 kPa for pressure diatomaceous earth filters, or a vacuum of –51 kPa for a vacuum system.

3. **Recirculation –** A recirculation or holding pump shall be employed to maintain differential pressure across the filter when the unit is not in operation in order to prevent the filter cake from dropping off the filter elements. A minimum recirculation of 0.24 m/hr shall be provided.

4. **Septum or Filter Element –** The filter elements shall be structurally capable of withstanding maximum pressure and velocity variations during filtration and backwash cycles, and shall be spaced such that no less than 25 mm is provided between elements or between any element and a wall.

5. **Inlet Design –** The filter influent shall be designed to prevent scour of the diatomaceous earth from the filter element.

3.3.2.3.9. Backwash

A satisfactory method to thoroughly remove and dispose of spent filter cake shall be provided.

3.3.2.3.10. Appurtenances

The following shall be provided for every filter:

1. Sampling taps for raw water and filtered water;

2. Loss of head or differential pressure gauge;

3. Rate-of-flow indicator, preferably with totalizer;

4. A throttling valve used to reduce rates below normal during adverse raw water conditions;

5. Evaluation of the need for body feed, recirculation, and any other pumps, in accordance with Section 3.5.3; and

6. Provisions for filtering to waste with appropriate measures for backflow prevention (see Section 3.3.12).

3.3.2.3.11. Monitoring

1. A continuous monitoring turbidimeter with recorder is required on the filter effluent for plants treating surface water.

2. Particle monitoring equipment should be provided as a means to enhance overall treatment operations for plants treating surface water.

3.3.2.4. Slow Sand Filters

The use of these filters shall require prior engineering studies to demonstrate the adequacy and suitability of this method of filtration for the specific raw water supply.

3.3.2.4.1. Quality of Raw Water

Slow rate gravity filtration shall be limited to waters having maximum turbidity of 10 units and maximum colour of 15 units; such turbidity must not be attributable to colloidal clay. Raw water quality data must include examinations for algae.

3.3.2.4.2. Number of Units

At least two units shall be provided. Where only two units are provided, each shall be capable of meeting the plant design capacity (normally the projected maximum daily demand) at the approved filtration rate. Where more than two filter units are provided, the filters shall be capable of meeting the plant design capacity at the approved filtration rate with one filter removed from service.

3.3.2.4.3. Structural Details and Hydraulics

Slow rate gravity filters shall be designed to provide:

1. A cover;

2. Headroom to permit normal movement by operating personnel for scraping and sand removal operations;
3. Adequate access hatches and access ports for handling of sand and for ventilation;
4. Filtration to waste;
5. An overflow at the maximum filter water level; and
6. Protection from freezing.

3.3.2.4.4. Rates of Filtration

The permissible rates of filtration shall be determined by the quality of the raw water and shall be on the basis of experimental data derived from the water to be treated. The normal rate may be 0.04 to 0.40 m/hr, with somewhat higher rates acceptable when demonstrated to the satisfaction of the DOEC.

3.3.2.4.5. Underdrains

Each filter unit shall be equipped with a main drain and an adequate number of lateral underdrains to collect the filtered water. The underdrains shall be so spaced that the maximum velocity of the water flow in the underdrain will not exceed 0.325 m/s. The maximum spacing of laterals shall not exceed 0.9 m if pipe laterals are used.

3.3.2.4.6. Filtering Material

1. Filter sand shall be placed on graded gravel layers for a minimum depth of 0.75 m.
2. The effective size shall be between 0.15 mm and 0.30 mm. Larger sizes may be considered by the DOEC; a pilot study may be required
3. The uniformity coefficient shall not exceed 2.5.
4. The sand shall be cleaned and washed free from foreign matter.
5. The sand shall be re-bedded when scraping has reduced the bed depth to no less than 0.475 m. Where sand is to be reused in order to provide biological seeding and shortening of the ripening process, re-bedding shall utilize a “throw over” technique whereby new sand is placed on the support gravel and existing sand is replaced on top of the new sand.

3.3.2.4.7. Filter Gravel

The supporting gravel should be similar to the size and depth distribution provided for rapid rate gravity filters (See Section 3.3.2.1.6. (5.e & f)).

3.3.2.4.8. Depth of Water on Filter Beds

Design shall provide a depth of at least 0.9 to 1.8 m of water over the sand. Influent water shall not scour the sand surface.

3.3.2.4.9. Control Appurtenances

Each filter shall be equipped with:

1. Loss of head gauge;
2. An orifice, Venturi meter, or other suitable means of discharge measurement installed on each filter to control the rate of filtration; and

3. An effluent pipe designed to maintain the water level above the top of the filter sand.

3.3.2.4.10. Ripening

Slow sand filters shall be operated to waste after scraping or re-bedding during a ripening period until the filter effluent turbidity falls to consistently below 1 NTU.

3.3.2.5. Direct Filtration

Direct filtration, as used herein, refers to the filtration of surface water following chemical coagulation and possibly flocculation but without prior settling. The nature of the treatment process will depend upon the raw water quality. A full-scale direct filtration plant shall not be constructed without prior pilot studies, which are acceptable to the DOEC. In-plant demonstration studies may be appropriate where conventional treatment plants are converted to direct filtration. Where direct filtration is proposed, an engineering report shall be submitted prior to conducting pilot plant or in-plant demonstration studies.

3.3.2.5.1. Technical Report

In addition to the items considered in Section 2.1, the Technical Report should include a historical summary of meteorological conditions and of raw water quality with special reference to fluctuations in quality, and possible sources of contamination. The following raw water parameters should be evaluated in the report:

1. Colour;
2. Turbidity;
3. Bacterial concentration;
4. Microscopic biological organisms;
5. Temperature;
6. Total solids;
7. General inorganic chemical characteristics; and
8. Additional parameters as required by the DOEC.

The report should also include a description of methods and work to be done during the pilot plant studies or, where appropriate, an in-plant demonstration studies.

3.3.2.5.2. Pilot Plant Studies

After approval of the Technical Report, pilot studies or in-plant demonstration studies shall be conducted. The studies must be conducted over a sufficient time to treat all expected raw water conditions throughout the year. The studies shall emphasize, but not be limited to, the following items:

1. Chemical mixing conditions including shear gradients and detention periods;
2. Chemical feed rates;

3. Use of various coagulants and coagulant aids;
4. Flocculation conditions;
5. Filtration rates;
6. Filter gradation, types of media and depth of media;
7. Filter breakthrough conditions; and
8. Adverse impact of recycling backwash water due to solids, algae, trihalomethane formation, and similar problems.

Prior to the installation of design plans and specifications, a final report including the engineer's design recommendations shall be submitted to the DOEC.

The pilot plant filter must be of a similar type and operated in the same manner as proposed for full-scale operation.

The pilot studies must demonstrate the minimum contact time necessary for optimum filtration for each coagulant proposed.

3.3.2.5.3. Pretreatment – Rapid Mix and Flocculation

The final rapid mix and flocculation basin design should be based on the pilot plant or in-plant demonstration studies augmented with applicable portions of Section 3.3.1.2 and Section 3.3.1.3.

3.3.2.5.4. Filtration

1. Filters shall be rapid rate gravity filters with dual or mixed media. The final filter design shall be based on the pilot plant or in-plant demonstration studies and all portions of Section 3.3.2.1. Pressure filters or single media sand filters shall not be used.

2. A continuous recording turbidimeter shall be installed on each filter effluent line and on the composite filter effluent line.

3. Additional continuous monitoring equipment to assist in control of coagulant dose may be required by the DOEC.

3.3.2.5.5. Siting Requirements

The plant design and land ownership surrounding the plant shall allow for the installation of conventional sedimentation basins should it be found that such are necessary.

3.3.2.6. Deep Bed Rapid Rate Gravity Filters

1. Deep bed rapid rate gravity filters, as used herein, generally refers to rapid rate gravity filters with filter material depths greater than 1.2 m. Filter media sizes are typically larger than those listed in Section 3.3.2.1.6. (5).

2. Deep bed rapid rate filters may be considered based on pilot studies pre-approved by the DOEC.

3. The final filter design shall be based on the pilot plant studies and shall comply with all applicable portions of Section 3.3.2.1. Careful attention shall be paid to the design of the backwash system which usually includes simultaneous air scour and water backwash at subfluidization velocities.

3.3.2.7. Biologically Active Filters

1. Biologically active filtration, as used herein, refers to the filtration of surface water (or ground water with iron, manganese or significant natural organic material), which includes the establishment and maintenance of biological activity within the filtration media.

2. Objectives of biologically active filtration may include control of disinfection by-products (DBPs), increased disinfection stability, reduction of substrates for microbial re-growth, breakdown of small quantities of synthetic organic chemicals, reduction of ammonia-nitrogen, and oxidation of iron and manganese. Biological activity can have an adverse impact on turbidity, particle or microbial pathogen removal, disinfection practices, head loss development, filter run times and distribution system corrosion. Design and operation should ensure that aerobic conditions are maintained at all times. Biologically active filtration often includes the use of ozone as a pre-oxidant/disinfectant, which breaks down natural organic materials into biodegradable organic matter, and granular activated carbon filter media, which may promote denser biofilms.

3. Biologically active filters may be considered based on pilot studies pre-approved by the DOEC. The study objectives must be clearly defined and must ensure the microbial quality of the filtered water under all anticipated conditions of operation. The pilot study shall be of sufficient duration to ensure establishment of full biological activity; often greater than three months is required.

4. The final filter design shall be based on the pilot plant studies and shall comply with all acceptable portions of Section 3.3.2.1.

3.3.2.8. Membrane Filtration for Treating Surface Sources

Low pressure membrane filtration technology has emerged as a viable option for addressing current and future drinking water regulations related to treatment of surface water sources. Recent research and applied full-scale facilities have demonstrated the efficient performance of both Micro filtration (MF) and Ultra filtration (UF) as feasible treatment alternatives to traditional granular media processes. Both MF and UF have been shown to be effective in removing identified parameters, such as, giardia/cryptosporidium, bacteria, turbidity and possibly viruses. The following provides a brief description of the characteristics of each process as well as selection and design considerations.

3.3.2.8.1. Characteristics

1. MF and UF membranes are most commonly made from organic polymers (for example, cellulose acetate, polysulfones, polyamides, polypropylene or polycarbonate). The physical configurations include hollow-fibre, spiral wound and tubular. MF membranes are capable

of removing particles with sizes down to 0.1 to 0.2 microns. UF processes have a probable lower cut-off rating of 0.005 to 0.1 microns.

2. Typical flux (rate of finished water permeate per unit membrane surface area) at 20EC for MF ranges between 20 to 41 m^3/m^2/day whereas the typical UF flux range is 4.0 to 20 m^3/m^2/day. Required operating pressures range from 35 to 70 kPa for MF and 100 to 500 kPa for UF.
3. Since both processes have relatively small membrane pore diameters, membrane fouling, caused by organic and inorganic as well as physical contaminants, is expected. Periodic flushing and cleaning is employed once a targeted transmembrane pressure differential has been reached. Typical cleaning agents utilized include acids, bases, surfactants, enzymes and certain oxidants, depending upon membrane material and foulants encountered.

4. Overall treatment requirements must be discussed with, and approved by, the DOEC. Disinfection is required with membrane filtration.

3.3.2.8.2. Selection and Design Considerations

1. A review of historical source raw water quality data, including turbidity and/or particle counts, organic loading, temperature differentials as well as other inorganic and physical parameters, can indicate whether either process is feasible. The degree of pretreatment, if any, may also be ascertained. Design consideration and membrane selection at this phase must also address the issue of target removal efficiencies versus acceptable transmembrane pressure differentials.

2. The useful life expectancy of a particular membrane under consideration should be evaluated. A membrane replacement frequency is a significant factor in operation and maintenance cost comparisons in the selection of the process.

3. Many membrane materials are incompatible with certain oxidants. If the system must rely on pre-treatment oxidants for other purposes, for example, zebra mussel control, taste and odour control, the selection of the membrane material becomes a significant design consideration.

4. The source water temperature can significantly impact the flux of the membrane under consideration. At low water temperatures the flux can be reduced appreciably, possibly impacting process feasibility or the number of membrane units required for a full-scale facility.

5. Flushing volumes can range from 5 to 25% of the permeate flow, depending upon the frequency of flushing/cleaning and the degree of fouling, and is an important factor in specifying the number of treatment units required.

6. An appropriate level of finished water monitoring should be provided to routinely evaluate membrane and housing integrity and overall filtration performance. Monitoring options may include particle counters, manual and/or automated pressure testing or air diffusion testing.

7. Cross connection considerations are necessary, particularly with regard to chemical feeds

used for membrane cleaning.

8. Redundancy of critical control components must be considered in the final design.

9. Other post-membrane treatment requirements must be evaluated in the final design to address other contaminants of concern such as colour and disinfection by-product precursors.

10. Prior to initiating the design of a membrane treatment facility, the DOEC should be contacted to determine if a pilot plant study would be required. In most cases, a pilot plant study will be necessary to determine the best membrane to use, particulate/organism removal efficiencies, cold and warm water flux, the need for pre-treatment, fouling potential, operating and transmembrane pressure and other design considerations. The DOEC should be contacted prior to conducting the pilot study to establish the protocol to be followed.

3.3.2.9. Reverse Osmosis

Reverse osmosis is a physical process in which suitably pretreated water is delivered at high pressure against a semi-permeable membrane. The membrane rejects most solute ions and molecules, while allowing water of very low mineral content to pass through. The process produces a reject concentrate waste stream in addition to the clear permeate product. Reverse osmosis systems have been successfully applied to saline groundwaters, brackish waters, and seawater.

The following items should be considered in evaluating the applicability for reverse osmosis:

1. **Membrane Selection -** Two types of membranes are typically used. These are Cellulose Acetate and Polyamide/Composite. Membrane configurations include tubular, spiral wound and hollow fine fibre. Operational conditions and useful life vary depending on the type of membrane selected.

2. **Useful Life of the Membrane** - The membrane represents a major cost component in the overall water system. Membrane replacement frequency can significantly affect the overall cost of operating the treatment facility.

3. **Pretreatment Requirements** - Acceptable feedwater characteristics are dependent on the type of membrane and operational parameters of the system. Without pretreatment, the membrane may become severely fouled and shorten its useful life. Pretreatment may be needed for turbidity reduction, iron or manganese removal, stabilization of the water to prevent scale formation, microbial control, chlorine removal, dissolved solids reduction, pH adjustment or hardness reduction.

4. **Treatment Efficiency** - Reverse osmosis is highly efficient in removing metallic salts and ions from the raw water. Efficiencies, however, do vary depending on the ion being removed and the membrane utilized. For most commonly encountered ions, removal efficiencies will range from 85% to over 99%. Organics removal is dependent on the molecular weight, the shape of the organic molecule and the pore size of the membrane utilized. Removal efficiencies may range from as high as 99% to less than 30%.

5. **Bypass Water** - Reverse osmosis permeate will be virtually demineralised. The design should provide for a portion of the raw water to bypass the unit to maintain stable water within the distribution system.

6. **Post Treatment** - Post treatment typically includes degasification for carbon dioxide and hydrogen sulphide removal (if present), pH adjustment for corrosion control and chlorination.

7. **Reject Water** - Reject water may range from 25% to 50% of the raw water pumped to the reverse osmosis unit. This may present a problem both from the source availability and from the waste treatment capabilities. The amount of reject water from a unit may be reduced, to a limited extent, by increasing the feed pressure to the unit, however this may result in a shorter membrane life. Acceptable methods of waste disposal include discharge to the municipal sewer system provided it satisfies the regulatory requirements of the DOEC or to an evaporation pond.

8. **Cleaning the Membrane** - The osmosis membrane must be replaced or periodically cleaned with acid. Method of cleaning, and chemicals used must be approved by the DOEC. Care must be taken in the acid cleaning process to prevent contamination of both the raw and finished water system.

9. **Pilot Plant Study** - Prior to initiating the design of a reverse osmosis treatment facility, the DOEC should be contacted to determine if a pilot plant study would be required. In most cases, a pilot plant study will be required to determine the best membrane to use, the type of pretreatment, type of post treatment, the bypass ratio, the amount of reject water, process efficiency and other design criteria.

10. **Operator training and Start-up** - The ability to obtain qualified operators must be evaluated in selection of the treatment process. The necessary operator training shall be provided prior to plant start-up.

3.3.2.10. Bag and Cartridge Filters

Bag and cartridge technology has been used for some time in the food, pharmaceutical and industrial applications. This technology is increasingly being used by small public water supplies for treatment of drinking water.

The particulate loading capacity of these filters is low, and once expended the bag or cartridge filter must be discarded. This technology is designed to meet the low flow requirement needs of small systems. The operational and maintenance cost of bag and cartridge replacement must be considered when designing a system. These filters can effectively remove particles from water in the size range of Giardia cysts (5 to 10 microns) and Cryptosporidium (2 to 5 microns).

At the present time, filtration evaluation is based on Giardia cyst removal. However, consideration should be given to the bag or cartridge filter ability to remove particles in the size range of Cryptosporidium since this is a current public health concern.

With this type of treatment there is no alteration of water chemistry. Therefore, once the technology has demonstrated the 2-log removal efficiency, no further pilot demonstration is necessary. The demonstration of filtration is specific to a particular housing and a particular bag or cartridge filter. Any other combinations of different bags, cartridges, or housings will require additional demonstration of filter efficiency.

Treatment of surface water should include source water protection, filtration, and disinfection.

The following sub-sections should be considered in evaluating the applicability of bag or cartridge filtration.

3.3.2.10.1. Predesign/Design

1. The filter housing and bag/cartridge filter must demonstrate a filter efficiency of 2-log reduction in particles sized 2 micron and above. The DOEC will decide whether or not a pilot demonstration is necessary for each installation. This filtration efficiency may be accomplished by:

    a. Macroscopic particulate analysis, including particle counting, sizing and identification, which determines occurrence and removals of micro-organisms and other particles across a filter or system under ambient raw water source condition, or when artificially challenged.

    b. Giardia/Cryptosporidium surrogate particle removal evaluation in accordance with procedures specified in NSF Standard 53 or equivalent. These evaluations can be conducted by NSF or by another third-party whose certification would be acceptable to the DOEC.

    c. “Nonconsensus” live Giardia challenge studies that have been designed and carried out by a third-party agent recognized and accepted by the DOEC for interim evaluations. At the present time uniform protocol procedures for live Giardia challenge studies have not been established. If a live Giardia challenge study is performed on site there must be proper cross-connection control equipment in place and the test portion must be operated to waste.

    d. Methods other than these that are approved by the DOEC.

2. System components such as housing, bags, cartridges, membranes, gaskets, and O-rings should be evaluated under NSF Standard 61 or equivalent, for leaching of contaminants. Additional testing may be required by the DOEC.

3. The source water or pretreated water should have turbidity less than 5 NTU.

4. It is recommended that the flow rate through the treatment process be monitored. The flow rate through the bag/cartridge filter must not exceed 76 L/min, unless documentation at higher flow rates demonstrates that it will meet the requirement for removal of particles.

5. Pretreatment is strongly recommended. This will provide a more constant water quality to the bag/cartridge filter. Examples of pretreatment include media filter, larger opening bag/cartridge filter, infiltration galleries, and beach wells. Location of the water intake should be considered in the pretreatment evaluation.

6. Particle count analysis can be used to determine what level of pretreatment should be provided. It should be noted that particulate counting is a ‘snap shot’ in time and that there can be seasonal variations such as algae blooms, lake turnover, spring runoff, and heavy rainfall events that will give varied water quality.

7. It is recommended that chlorine or another disinfectant be added at the head of the treatment process to reduce/eliminate the growth of algae, bacteria, etc., on the filters. The impact on disinfection by-product formation should be considered.

8. A filter to waste component is strongly recommended, for any pretreatment pressure sand filters. At the beginning of each filter cycle and/or after every backwash of the pre-filters a set amount of water should be discharged to waste before water flows into the bag/cartridge filter.

9. If pressure media filters are used for pretreatment they must be designed according to Section 3.3.2.2.

10. A sampling tap shall be provided ahead of any treatment so a source water sample can be collected.

11. Pressure gauges and sampling taps shall be installed before and after the media filter, and before and after the bag/cartridge filter.

12. An automatic air release valve shall be installed on top of the filter housing.

13. Frequent start and stop operation of the bag or cartridge filter should be avoided. To avoid this frequent start and stop cycle the following options are recommended:

    a) A slow opening and closing valve ahead of the filter to reduce flow surges;

    b) Reduce the flow through bag or cartridge filter to as low as possible to lengthen filter run times; and

    c) Install a re-circulating pump that pumps treated water back to a point ahead of the bag or cartridge filter. Care must be taken to ensure there is no cross connection between the finished water and raw water.

14. A minimum of two bag or cartridge filter housings should be provided for water systems that must provide water continuously.

15. A pressure relief valve should be incorporated into the bag or cartridge filter housing.

16. Complete automation of the treatment system is not required. Automation of the treatment plant should be incorporated into the ability of the water system to monitor the finished water quality. It is important that a qualified water operator is available to run the treatment plant.

17. A plan of actions should be in place should the water quality parameters fail to meet the current requirements of the DOEC with respect to microbiological, physical, chemical and radiological qualities.

3.3.2.10.2. Operations

1. The filtration and backwash rates shall be monitored so that the pre-filters are being optimally used.

2. The bag and cartridge filters must be replaced when a pressure difference of 210 kPa or other pressure difference recommended by the manufacturer is observed. It should be noted that bag filters do not load linearly. Additional observation of the filter performance is required near the end of the filter fun.

3. Maintenance (o-ring replacement) shall be performed in accordance with the manufacturer's recommendations.

4. The following parameters should be monitored:
   a) Instantaneous flow rate;
   b) Total flow rate;
   c) Operating pressure;
   d) Pressure differential; and
   e) Turbidity.

3.3.3. Softening

Common processes for conventional water softening include lime, lime with soda ash, ion exchange, and combinations of these methods.

The softening process selected must be based upon the mineral qualities of the raw water and the desired finished water quality, in conjunction with requirements for disposal of sludge or brine waste, cost of plant, cost of chemicals and plant location. Applicability of the process chosen shall be demonstrated.

3.3.3.1. Lime or Lime - Soda Process

The process involves the thorough mixing of the chemicals with the water, followed by slow agitation to allow completion of the chemical reaction. Stabilization of the softened water is then required. This process cannot be expected to produce water with much less than 80 mg/L of hardness.

3.3.3.1.1. Chemicals
Either hydrated lime or quick lime is usually used in water softening, the choice depending on the availability of the chemicals and required equipment, together with the cost. Lime and recycled sludge should be fed directly into the rapid mix basin.

3.3.3.1.2. Reaction Basin
Designs standards for the reaction basin are as per those contained in Section 3.3.1.3.

Rapid mix basins must provide more than 30 seconds detention time with adequate velocity gradients to keep the lime particles dispersed.

3.3.3.1.3. Hydraulics
When split treatment is used, the bypass line should be sized to carry total plant flow, and an accurate means of measuring and splitting the flow must be provided.

3.3.3.1.4. Aeration
Determinations should be made for the carbon dioxide content of the raw water. When concentrations exceed 10 mg/L, the economics of removal by aeration as opposed to removal with lime should be considered if it has been determined that dissolved oxygen in the finished water will not cause corrosion problems in the distribution system. (See Section 3.3.4)

3.3.3.1.5. Stabilization
In the softening process the addition of lime to water reduces the carbonate saturation index and increases the tendency of the water to deposit calcium carbonate. The carbonate balance may be partly or completely restored by stabilization of the softened water. This can be achieved by the addition of carbon dioxide, acid or polyphosphates.

1. **Carbon Dioxide:**

   For treated waters with pH over 9.5 during treatment, two-stage recarbonation equipment should be considered. Single stage recarbonation ahead of filtration will probably be sufficient for treated waters with a pH below 9.5. The recarbonation basin should have a water depth of approximately 2.5 m and provide a detention time of between 3 and 10 min.

   Adequate precaution must be taken to prevent the possibility of carbon monoxide entering the plant from recarbonation compartments. Adequate ventilation should be provided.

2. **Acid:**

   Acid should be fed in concentrated solution ahead of the filters. Feed equipment should be located as close to the point of application as possible. Adequate precautions shall be taken for safety, such as not adding water to the concentrated acid.

3. **Polyphosphates:**

   Several types of polyphosphates, containing variable amounts of alkali are available for stabilization. The stock solutions should be kept covered, and should contain satisfactory chlorine residuals of approximately 10 mg/L.

3.3.3.1.6. Sludge Collection and Disposal
Mechanical sludge removal equipment shall be provided in the sedimentation basin, and sludge recycling to the rapid mix should be provided.

Provision must be included for the proper disposal of softening sludge (see Section 3.3.12)

3.3.3.1.7. Disinfection
The use of excess lime shall not be considered an acceptable substitute for disinfection (See Section 4)

3.3.3.1.8. Plant Start-up
The plant processes must be manually started following shutdown.

3.3.3.2. Cation Exchange Process

The mineral quality of the raw water must be considered when softening by this method; the process does not reduce the total solids content but merely substitutes sodium in the hardness-causing compounds.

Alternative methods of hardness reduction should be investigated when the sodium content and dissolved solids concentration is of concern. Iron, manganese, or a combination of the two, should not exceed 0.3 mg/L in the water as applied to the ion exchange resin.

Waters having turbidity of 5 NTUs or more should not be applied directly to cation-exchange softeners. Waters with a pH above 8.4 should not be applied to silica gel materials. Waters containing less than 12 mg/L of silica as silicon dioxide should not be applied to siliceous cation material. When the applied water contains a chlorine residual, the ion-exchange materials should be of a type that is not damaged by residual chlorine. Phenolic resin exchange materials shall not be used unless approved methods are utilized for disinfecting the material prior to use.

3.3.3.2.1. Design
Where bacterial removal is not involved, ion-exchange softeners may be of either the pressure or open gravity type. When open gravity units are used, the water should be chlorinated. Either upflow or downflow units may be used. Automatic regeneration is highly desirable when only part-time attendance is to be provided. Raw waters containing turbidity, suspended matter, dissolved iron or manganese, high concentrations of certain salts, and particularly any substances deleterious to the exchange materials, should be treated before softening.

3.3.3.2.2. Capacity
The design capacity for hardness removal should not exceed 48,000 mg/L when resin is regenerated with 2.l kg of salt per l kg of hardness removed.

3.3.3.2.3. Depth of Resin
The depth of the exchange resin should not be less than 0.9 m.

3.3.3.2.4. Rate of Flow
The rate of flow through a softening unit should not exceed 17 m/hr. The rate of backwash should be between 14 to 20 m/hr of bed area. Rate-of-flow controllers, or the equivalent, must be installed for the above purposes.

3.3.3.2.5. Freeboard
The freeboard will depend on the specific gravity of the resin and the direction of water flow. Generally, the washwater collector should be 0.6 m above the top of the resin on downflow units.

3.3.3.2.6. Underdrains and Supporting Gravel
The bottoms, strainer systems and support for the exchange resin should conform to the criteria provided for rapid rate gravity filters (see Section 3.3.2.1).

3.3.3.2.7. Distribution of Brine
Facilities should be included for even distribution of the brine over the entire surface of both upflow and downflow units.

3.3.3.2.8. Cross-connection Control
Backwash, rinse and air relief discharge pipes should be installed in such a manner as to prevent any possibility of back-siphonage.

3.3.3.2.9. Bypass
A bypass shall be provided around all softening units to produce a blended water of the desired hardness. Totalizing meters should be installed on the bypass line and on each softener unit. An automatic proportioning or regulating device, and shut-off valve should be provided on the bypass line. In some installations, it may be necessary to treat the bypassed water to obtain acceptable levels of iron and/or manganese in the finished water.

3.3.3.2.10. Additional Limitations
Silica gel resins should not be used for waters having a pH above 8.4 or containing less than 6.0 mg/L silica and should not be used when iron is present. When the applied water contains a chlorine residual, the cation exchange resin shall be a type that is not damaged by residual chlorine. Phenolic resin should not be used.

3.3.3.2.11. Wet Salt Storage Tanks and Brine Tanks
1. Wet salt storage and salt dissolving or brine tanks must be covered, corrosion resistant, and be equipped with manhole or hatchway openings having raised curbs and watertight covers with overlapped edges similar to those required for finished water reservoirs.

2. Overflow pipes should be turned downward, have a free fall discharge or a self-closing flap valve, and be covered with a small corrosion resistant mesh screen.

3. Wet salt storage tanks should have sufficient capacity to store 1.5 carloads or 1.5 truckloads (depending on method of delivery) of salt in order to permit refill before a tank is completely empty. Alternately, two wet salt storage tanks or compartments designed to operate independently should be provided.

4. Water for filling a tank should be distributed over the entire surface of the tank by pipes above the maximum brine level in the tank. The salt should be supported on graduated layers of gravel under which there is half tile or other suitable means of collecting the brine.

5. Salt storage capacity should generally be adequate for at least one month.

6. A brine-measuring tank should have a capacity in excess of that required for regeneration of one unit.

7. An injector may be used to transfer brine from the brine tank to the softeners. If a pump is used, a brine-measuring tank or means of metering should be provided to obtain proper dilution.
8. Consideration should be given to the advisability of disinfecting the brine when the quality of the salt may be inferior, or the method of handling is questionable.

3.3.3.2.12. Stabilization
The need for corrective treatment should be determined. Soda ash, caustic soda, caustic silicate, polyphosphates or a combination of these, or other alkali may be added to the softened (or blended) water. Positive displacement solution feed pumps, coordinated with the softening units, should be used.

3.3.3.2.13. Sampling Taps
Smooth-nosed sampling taps should be provided for the collection of representative samples for both bacteriological and chemical analyses. The taps should be located to provide for sampling of the softener influent and of the blended water when any hard water is bypassed. The blended water-sampling tap should be at least 6 m downstream from the point of blending in order to assure a representative sample. Petcocks are not acceptable as sampling taps. Sampling taps should be provided on the brine tank discharge piping.

3.3.3.2.14. Waste Disposal
The DOEC shall be consulted, and its approval obtained concerning the disposal of brine wastes.

3.3.3.2.15. Construction Materials
Pipes and contact materials must be resistant to the aggressiveness of salt. Plastic and red brass are acceptable piping materials. Steel and concrete must be coated with a non-leaching protective coating, which is compatible with salt and brine.

3.3.3.2.16. Housing
Bagged salt and dry bulk salt storage shall be enclosed and separated from other operating areas in order to prevent damage to equipment.

3.3.3.3. Water Quality Test Equipment
Test equipment for alkalinity, total hardness, carbon dioxide content, and pH should be provided to determine treatment effectiveness.

3.3.4. Aeration

Aeration may be used to reduce or remove objectionable amounts of carbon dioxide, hydrogen sulphide, methane, tastes and odours, and to introduce oxygen to assist in iron and/or manganese removal. Local conditions, the quality of the water to be treated, and the cost should be carefully considered before deciding on the type of aeration. The packed tower aeration process is an aeration process applicable to removal of volatile organic contaminants.

3.3.4.1. Natural Draft Aeration

The design shall provide:

1. Discharge through a series of three or more trays with separation of the trays between 300 and 375 mm;
2. Perforations in the distribution pan that are 5 mm to 12 mm in diameter and spaced at 25 mm to 75mm on centres to maintain a 250 mm water depth;
3. Media, when used on the trays, should be structurally sound crushed rock, slag or specially manufactured material, from 75 mm to 100 mm in size;
4. For distribution of water uniformly over the top tray;
5. Loading at a rate of 0.68 to 3.4 L/sec/m^2 of total tray area;
6. Trays with slotted, heavy wire (12 mm openings) mesh or perforated bottoms;
7. Construction of durable material resistant to aggressiveness of the water and dissolved gases;
8. Protection from loss of spray water by wind carriage by enclosure with louvers sloped to the inside at an angle of approximately 45 degrees; and
9. Protection from insects by 24-mesh screen.

3.3.4.2. Forced or Induced Draft

Devices shall be designed to provide the following:

1. An insect and light proof enclosure constructed of steel, wood, or other durable material;
2. A ventilating blower or fan having a weatherproof motor in a tight housing with a screened air inlet. The air outlet of the aerator should be turned down and screened;
3. Wood slats, trays or other devices should be located in the enclosure to provide adequate water distribution for air contact;
4. Aerator can be easily reached or removed for maintenance of the interior, or installed in a separate aerator room;

5. Aerator to be located in an area as free as possible from obnoxious fumes, dust and dirt;

6. Provide loading at a rate of 0.68 to 3.4 L/sec/m^2 of total tray area;

7. Ensure that the water outlet is adequately sealed to prevent unwarranted loss of air;

8. Discharge through a series of five or more trays with separation of trays not less that 150 mm;

9. Distribution of water uniformly over the top tray; and

10. Be of durable material resistant to the aggressiveness of the water and dissolved gases.

3.3.4.3. Spray Aeration

Design shall provide:

1. A hydraulic head of between 1.5 and 7.5 m;

2. Nozzles, with the size, number, and spacing of the nozzles being dependent on the flowrate, space, and the amount of head available;

3. Nozzle diameter in the range of 12 to 17 mm to minimize clogging, and

4. An enclosed basin to contain the spray. Any opening for ventilation, etc. must be protected with a 24-mesh screen.

3.3.4.4. Pressure Aeration

Pressure aeration may be used for oxidation purposes only if pilot plant study indicates the method is applicable. It is not acceptable for removal of dissolved gases. Filters following pressure aeration must have adequate exhaust devices for release of air. Pressure aeration devices shall be designed to:

1. Give thorough mixing of compressed air with water being treated, and

2. Provide screened and filtered air, free of obnoxious fumes, dust, dirt and other contaminants.

3.3.4.5. Packed Tower Aeration

Packed tower aeration (PTA), which is also known as air stripping, involves passing water down through a column of packing material while pumping air counter-currently up through the packing. PTA is used for the removal of volatile organic chemicals, THMs, carbon dioxide, and radon. Generally, PTA is feasible for compounds with a Henry’s Constant greater than 100 (expressed in atm mol/mol) - 12º C), but is not normally feasible for removing compounds with a Henry’s Constant less than 10. For values between 10 and 100, PTA may be feasible but should be extensively evaluated using pilot studies. Values for Henry’s Constant should be discussed with the DOEC prior to final design.

3.3.4.5.1. Process Design

1. Process design methods for PTA involve the determination of Henry's Constant for the contaminant, the mass transfer coefficient, air pressure drop and stripping factor. The applicant shall provide justification for the design parameters selected (i.e. height and diameter of unit, air to water ratio, packing depth, surface loading rate, etc.). Pilot plant testing shall be provided. The pilot test shall evaluate a variety of loading rates, and air to water ratios at the peak contaminant concentration. Special consideration should be given to removal efficiencies when multiple contaminations occur. Where there is considerable past performance data on the contaminant to be treated and there is a concentration level similar to previous projects, the DOEC may approve the process design based on use of appropriate calculations without pilot testing. Proposals of this type must be discussed with the DOEC prior to the submission of any permit applications.

2. The tower shall be designed to reduce contaminants to below the maximum contaminant level (MCL) and to the lowest practical level.

3. The ratio of the column diameter to packing diameter should be at least 7:1 for the pilot unit and at least 10:1 for the full-scale tower. The type and size of the packing used in the full-scale unit shall be the same as that used in the pilot work.

4. The minimum volumetric air to water ratio at peak water flow should be 25:1. The maximum air to water ratio for which credit will be given is 80:1.

5. The design should consider potential fouling problems from calcium carbonate and iron precipitation and from bacterial growth. It may be necessary to provide pretreatment. Disinfection capability shall be provided prior to and after PTA.

6. The effects of temperature should be considered since a drop in water temperature can result in a drop in contaminant removal efficiency.

3.3.4.5.2. Materials of Construction

1. The tower can be constructed of stainless steel, concrete, aluminum, fibreglass or plastic. Uncoated carbon steel is not recommended because of corrosion. Towers constructed of lightweight materials should be provided with adequate support to prevent damage from wind.

2. Packing materials shall be resistant to the aggressiveness of the water, dissolved gases and cleaning materials, and shall be suitable for contact with potable water.

3.3.4.5.3. Water Flow System

1. Water should be distributed uniformly at the top of the tower using spray nozzles or orifice-type distributor trays that prevent short-circuiting. For multi-point injection, one injection point for every 190 cm^2 of tower cross-sectional area is recommended.

2. A mist eliminator shall be provided above the water distribution system.

3. A side wiper redistribution ring should be provided at least every 3 m in order to prevent water channelling along the tower wall and short-circuiting.

4. Sample taps shall be provided in the influent and effluent piping.

5. The effluent sump, if provided, shall have easy access for cleaning purposes and be equipped with a drain valve. The drain shall not be connected directly to any storm or sanitary sewer.

6. A blow-off line should be provided in the effluent piping to allow for discharge of water/chemicals used to clean the tower.

7. The design shall prevent freezing of the influent riser and effluent piping when the unit is not operating. If piping is buried, it shall be maintained under positive pressure.

8. The water flow to each tower shall be metered.

9. An overflow line shall be provided with discharges located 300 to 350 mm above a splash pad or drainage inlet. Proper drainage shall be provided to prevent flooding of the area.

10. Butterfly valves may be used in the water effluent line for better flow control, as well as to minimize air entrainment.

11. Means shall be provided to prevent flooding of the air blower.

12. The water influent pipe should be supported separately from the tower's main structural support.

3.3.4.5.4. Air Flow System

1. The air inlet to the blower and the tower discharge vent shall be down turned and protected with a non-corrodible 24-mesh screen to prevent contamination from extraneous matter. It is recommended that a 4-mesh screen also be installed prior to the 24-mesh screen on the air inlet system.

2. The air inlet shall be in a protected location.

3. An air flow meter shall be provided on the influent air line, or an alternative method to determine the air flow shall be provided.

4. A positive air flow sensing device and a pressure gauge must be installed on the air influent line. The positive air flow sensing device must be a part of an automatic control system, which will turn off the influent water if positive air flow is not detected. The pressure gauge will serve as an indicator of fouling build-up.

5. A backup motor for the air blower must be readily available.

3.3.4.5.5. Other Features

Other features that shall be provided:

1. A sufficient number of access ports with a minimum diameter of 0.6 m to facilitate inspection, media replacement, media cleaning and maintenance of the interior.

2. A method of cleaning the packing material when iron, manganese, or calcium carbonate fouling may occur.

3. Tower effluent collection and pumping wells constructed to clearwell standards.

4. Provisions for extending the tower height without major reconstruction.

5. An acceptable alternative supply must be available during periods of maintenance and operation interruptions. No bypass shall be provided unless specifically approved by the DOEC.

6. Disinfection application points both ahead of and after the tower to control biological growth.

7. Disinfection and adequate contact time after the water has passed through the tower and prior to the distribution system.

8. Adequate packing support to allow free flow of water and to prevent deformation with deep packing heights.

9. Operation of the blower and disinfectant feeder equipment during power failures.

10. Adequate foundation to support the tower and lateral support to prevent overturning due to wind loading.

11. Fencing and locking gate to prevent vandalism.

12. An access ladder with safety cage for inspection of the aerator including the exhaust port and de-mister.

13. Electrical interconnection between blower, disinfectant feeder and well pump.

3.3.4.5.6. Environmental Factors

1. The applicant must contact the DOEC to determine if permits are required.

2. Noise control facilities should be provided on PTA systems located in residential areas.

3.3.4.6. Other Methods of Aeration

Other methods of aeration may be used if applicable to the treatment needs. Such methods include, but are not restricted to, spraying, diffused air, cascades and mechanical aeration. The treatment processes must be designed to meet the particular needs of the water to be treated and are subject to the approval of the DOEC.

3.3.4.7. Protection of Aerators

All aerators except those discharging to lime softening or clarification plants shall be protected from contamination by birds, insects, wind borne debris, rainfall and water draining off the exterior of the aerator.

3.3.4.8. General Design

3.3.4.8.1. Wind Protection

Spray aerators should be enclosed between walls or a louvered fence, the louvers being sloped down to the inside at an angle of approximately 45 degrees.

3.3.4.8.2. Disinfection

Groundwater supplies exposed to the atmosphere by aeration must receive chlorination as the minimum additional treatment.

3.3.4.8.3. Bypass

A bypass should be provided for all aeration units except those installed to comply with maximum contaminant levels.

3.3.4.8.4. Contamination

Waters, which do not require additional treatment, should be protected by a non-corrodible fine screen. A water, dirt, and dust tight roof should cover the aerator.

3.3.4.8.5. Corrosion Control

The aggressiveness of the water after aeration should be determined and corrected by additional treatment, if necessary.

Internal and external corrosion of a public water supply distribution system is a recognized problem that cannot be completely eliminated but can be effectively controlled. Aside from the economic and aesthetic problems, the possible adverse health effects of corrosion products, such as lead and copper, is a major consideration

Corrosion of metallic pipes is an electrochemical process by which the pipe material is chemically oxidized. This can occur as a result of heterogeneity of dissimilar metals, creating electrical potential or by the attacking of the interior walls of the pipe by aggressive molecules in the water.

Control of corrosion is a function of the design, maintenance, and operation of a public water supply. These functions must be considered simultaneously in order for the corrosion control program to function properly. Corrosion problems must be solved on an individual basis depending on the specific water quality characteristics and materials used in the distribution system. Specific information can be obtained from publications of technical agencies and associations such as USEPA (Lead and Copper Regulations, 1994) and the American Water Works Association (Lead and Copper Strategies, 1990; Chemistry of Corrosion Inhibitors in Potable Waters, 1990). Broad areas of consideration for a corrosion control program follow.

The following apply for internal corrosion:

1. Provide for a system of records by which the nature and frequency of corrosion problems are recorded. On a schematic of the distribution system, show the location of each problem so that follow-up investigations and improvements can be made when a cluster of problems is identified.

2. When complaints are received from a customer, follow up with an inspection by experienced personnel or consultant experienced in corrosion control. Where advisable, obtain samples of water using appropriate sampling protocols for chemical and microbiological analyses and piping and plumbing material samples. Analyses should be made to determine the type and, if possible, the cause of the corrosion.

3. Establish a program or conduct desktop analyses or loop studies to determine the corrosiveness/determination of the stability of the water in representative parts of the distribution system. Analysis for alkalinity, pH, temperature, and corrosion products (such as lead, cadmium, copper, and iron) should be performed on water samples collected at the treatment plant or wellhead, and at representative points in the distribution system. In comparing the analyses of the source water with the distribution system water, significant changes in alkalinity, pH, or corrosion products would indicate that corrosion is taking place, and thereby indicate that corrective steps need to be taken.

4. Where possible, especially when corrosion has been detected in the determination of water stability, provide a program that will measure both the physical and chemical aspects of the corrosion phenomena. Physical measurement of the rate of corrosion can be made by the use of coupons, easily removed sections of pipe, connected flow-through pipe test sections, or other piping using desktop analyses or corrosion indices such as the Langelier Index, Byznar Index, or Aggressiveness Index (AWWA C-400). Correlation of the data from the physical measurement with the data from the selected corrosion analysis will provide information to determine the type of corrective treatment needed and may allow for the subsequent use of the corrosion analysis alone to determine the degree of corrosivity in select areas of the distribution system.

5. If corrosion is found to exist throughout the distribution system, corrective measures at the treatment plant, pump station or wellhead should be initiated. A chemical feed can be made to provide a stable to slightly depositing water or water quality, which mitigates the solubility of targeted parameters. In calculating the stability index and the corresponding chemical feed adjustments, consideration must be given to items such as: the water temperature, if it varies with the season and within various parts of the distribution system; the velocity of flow within various parts of the distribution system; the degree of stability needed by the individual customer; and the dissolved oxygen content of distributed water, especially in water having low hardness and alkalinity. Threshold treatment involving the feeding of a ortho or blended phosphate or a silicate to control corrosion may be considered for both ground and surface water supplies.

6. Additional control of corrosion problems can be obtained by a regulation or ordinance for the materials used in or connected to a distribution system. Careful selection of material compatible with the physical system or the water being delivered can aid in reduction of corrosion product production.

Note: Adjustment of pH for corrosion control must not interfere with other pH dependent processes (e.g., colour removal by alum coagulation) or aggravate other water quality parameters (e.g., THM formation). In addition, the use of ortho- or blended phosphates should not aggravate distribution microbial concerns or adversely impact wastewater facilities.

3.3.4.8.6. Quality Control

Equipment should be provided to test for DO, pH, and temperature to determine proper functioning of the aeration device. Equipment to test for iron, manganese, and carbon dioxide should also be considered.

3.3.5. Iron And Manganese Removal

Iron and manganese control refers solely to the treatment processes designed specifically for this purpose. The treatment process used will depend upon the character of the raw water. The selection of one or more treatment processes must meet specific local conditions as determined by engineering investigations, including chemical analysis of representative samples of water to be treated, and receive the approval of the DOEC. It may be necessary to operate a pilot plant in order to gather all information pertinent to the design. Consideration should be given to adjusting pH of the raw water to optimize the chemical reaction. Testing equipment and sampling taps shall be provided as outlined in Sections 3.1.7 and 3.1.9

3.3.5.1. Removal by Oxidation, Detention and Filtration

3.3.5.1.1. Oxidation

Oxidation may be by aeration, as indicated in Section 3.3.4, or by chemical oxidation with chlorine, potassium permanganate, ozone or chlorine dioxide.

3.3.5.1.2. Detention

1. **Reaction –** A minimum detention time of 30 minutes shall be provided following aeration to ensure that the oxidation reactions are as complete as possible. This minimum detention may be omitted only when a pilot plant study indicates no need for detention. The detention basin may be designed as a holding tank without provisions for sludge collection but with sufficient baffling to prevent short-circuiting.

2. **Sedimentation** - Sedimentation basins shall be provided for treated water with high iron and/or manganese content, or where chemical coagulation is used to reduce the load on the filters. Provisions for sludge removal shall be made.

3.3.5.1.3. Filtration

Filters shall be provided and shall conform to Section 3.3.2.

3.3.5.2. Removal by Lime - Soda Softening Process

See Section 3.3.3.1.

3.3.5.3. Removal by Manganese Coated Media Filtration

This process consists of a continuous feed of potassium permanganate to the influent of a manganese coated media filter.

1. Provisions should be made to apply the permanganate as far ahead of the filter as practical and to a point immediately before the filter.

2. Other oxidizing agents or processes such as chlorination or aeration may be used prior to the permanganate feed to reduce the cost of the chemical.

3. An anthracite media cap of at least 150 mm shall be provided over manganese coated media.

4. Normal filtration rate is 7.2 m/hr.

5. Normal wash rate is 20 to 24 m^3/hr for manganese greensand and 37 to 49 m^3/hr for manganese coated media.

6. Air washing should be provided.

7. Sample taps shall be provided:
   a. Prior to application of permanganate;
   b. Immediately ahead of filtration;
   c. At the filter effluent; and
   d. At points between the anthracite media and the manganese coated media.

3.3.5.4. Removal by Units Regenerated with Potassium Permanganate

Iron and manganese, which can be oxidized by aeration, may be removed by passing the water through exchange media, regenerating the media with potassium permanganate. Pressure units may be used for this type of treatment. The rate through such units should not exceed 2.0 L/m^2s. Care should be taken not to aerate the water before it enters the exchange unit.

3.3.5.5. Removal by Ion Exchange

This process of iron and manganese removal should not be used for water containing more than 0.3 mg/L of iron, manganese or combination thereof. This process is not acceptable where either the raw water or wash water contains dissolved oxygen or other oxidants.

3.3.5.6. Sequestration by Polyphosphates

This process shall not be used when iron, manganese or combination thereof exceeds 1.0 mg/L. The total phosphate applied shall not exceed 10 mg/L as PO_4. Where phosphate treatment is used, satisfactory chlorine residuals shall be maintained in the distribution system. Possible adverse affects on corrosion must be addressed when phosphate addition is proposed for ion sequestering.

1. Feeding equipment shall conform to the requirements of Section 3.4.

2. Stock phosphate solution must be kept covered and disinfected by carrying approximately 10 mg/L free chlorine residual. Phosphate solutions having a pH of 2.0 or less may be exempted from this requirement by the DOEC.

3. Polyphosphates shall not be applied ahead of iron and manganese removal treatment. The point of application shall be prior to the aeration, oxidation or disinfection, if no iron or manganese removal treatment is provided.

3.3.5.7. Sequestration by Sodium Silicates

Sodium silicate sequestration of iron and manganese is appropriate only for groundwater supplies prior to air contact. On-site pilot tests are required to determine the suitability of sodium silicate for the particular water and the minimum feed needed. Rapid oxidation of the metal ions, such as by chlorine or chlorine dioxide, must accompany or closely precede the sodium silicate addition. Injection of sodium silicate more than 15 seconds after oxidation may cause detectable loss of chemical efficiency. Dilution of feed solutions much below 5% silica as SO_2 should also be avoided for the same reason.

1. Sodium silicate addition is applicable to waters containing up to 2 mg/L of iron, manganese or combination thereof.

2. Chlorine residuals shall be maintained throughout the distribution system to prevent biological breakdown of the sequestered iron.

3. The amount of silicate added shall be limited to 20 mg/L as SiO_2, but the amount of added and naturally occurring silicate shall not exceed 60 mg/L as SiO_2.

4. Feeding equipment shall conform to the requirements of Section 3.4.

5. Sodium silicate shall not be applied ahead of iron or manganese removal treatment.

3.3.5.8. Sampling Taps

Smooth-nosed sampling taps shall be provided for control purposes. Taps shall be located on each raw water source, each treatment unit influent, and each treatment unit effluent.

3.3.5.9. Testing Equipment

1. The equipment should have the capacity to accurately measure the iron content to a minimum of 0.1 mg/L, and the manganese content to a minimum of 0.05 mg/L.

2. Where polyphosphate sequestration is practiced, appropriate phosphate testing equipment shall be provided.

3.3.6. Nitrate Removal Using Sulphate Selective Anion Exchange Resin

Four treatment processes are generally considered acceptable for Nitrate/Nitrite removal. These are anion exchange, reverse osmosis, nanofiltration and electrodialysis. Although these treatment processes, when properly designed and operated, will reduce the nitrate/nitrite concentration of the water to acceptable levels, primary consideration shall be given to reducing the nitrate/nitrite levels of the raw water through either obtaining water from an alternate water source or through watershed management. Reverse osmosis, nanofiltration or electrodialysis should be investigated when the water has high levels of sulphate or when the chloride content or dissolved solids concentration is of concern.

Most anion exchange resins used for nitrate removal are sulphate selective resins. Although nitrate selective resins are available, these resins typically have a lower total exchange capacity.

3.3.6.1. Special Caution

If a sulphate selective anion exchange resin is used beyond bed exhaustion, the resin will continue to remove sulphate from the water by exchanging the sulphate for previously removed nitrates resulting in treated water nitrate levels being much higher than raw water levels. Therefore, it is extremely important that the system not be operated beyond design limitations.

3.3.6.2. Pretreatment Requirement

An evaluation shall be made to determine if pretreatment of the water is required if the combination of iron, manganese, and heavy metals exceeds 0.1 mg/L.

3.3.6.3. Design

Anion exchange units are typically of the pressure type, down flow design. Although a pH spike can typically be observed shortly before bed exhaustion, automatic regeneration based on volume of water treated should be used unless justification for alternate regeneration is submitted to and approved by the DOEC. A manual override shall be provided on all automatic controls. A minimum of two units must be provided. The total treatment capacity must be capable of producing the maximum daily water demand at a level below the nitrate/nitrite MCL. If a portion of the water is bypassed around the unit and blended with the treated water, the maximum blend ratio allowable must be determined based on the highest anticipated raw water nitrate level. If a bypass is provided, a totalling meter and a proportioning or regulating valves must be provided on the bypass line.

3.3.6.4. Exchange Capacity

Anion exchange media will remove both nitrates and sulphate from the water being treated. The design capacity for nitrate and sulphate removal expressed as $CaCO_3$ should not exceed 565 grains per litre when the resin is regenerated with 0.545 kg of salt per cubic foot (160) g/l) of resin when operating at 0.27 to 0.4 L/min/L. However, if high levels of chlorides exist in the raw water, the exchange capacity of the resin should be reduced to account for the chlorides.

3.3.6.5. Flow Rates

The treatment flow rate should not exceed 29 to 32 cm/min down flow rate. The back wash flow rate should be 8 to 12 cm/min, with a fast rinse approximately equal to the service flow rate.

3.3.6.6. Freeboard

Adequate freeboard must be provided to accommodate the backwash flow rate of the unit.

3.3.6.7. Miscellaneous Appurtenances

The system shall be designed to include an adequate under drain and supporting gravel system, brine distribution equipment, and cross connection control.

3.3.6.8. Monitoring

Whenever possible, the treated water nitrate/nitrate level should be monitored using continuous monitoring and recording equipment. The continuous monitoring equipment should be equipped with a high nitrate level alarm. If continuous monitoring and recording equipment is not provided, the finished water nitrate/nitrite levels must be determined (using a test kit) no less than daily, preferably just prior to regeneration of the unit.

3.3.6.9. Waste Disposal

Generally, waste from the anion exchange unit should be disposed in accordance with Section 3.3.12. However, prior to any discharge, the DOEC must be contacted for wastewater discharge limitation.

3.3.6.10. Additional Limitations

Certain types of anion exchange resins can tolerate no more than 0.05 mg/L of free chlorine. When the applied water will contain a chlorine residual, the anion exchange resin must be a type that is not damaged by residual chlorine.

3.3.7. Taste and Odour Control

Provision shall be made for the control of taste and odour at all surface water treatment plants. Chemicals shall be added sufficiently ahead of other treatment processes to ensure adequate contact time for an effective and economical use of the chemicals. Where severe taste and odour problems are encountered, in-plant and/or pilot plant studies are required.

3.3.7.1. Flexibility

Plants treating water that is known to have taste and odour problems should be provided with equipment that makes several of the control processes available so that the operator will have flexibility in operation.

3.3.7.2. Chlorination

Chlorination can be used for the removal of some objectionable odours. Adequate contact time must be provided to complete the chemical reactions involved. Excessive potential by-product production through this process should be avoided by adequate bench-scale testing prior to design.

3.3.7.3. Chlorine Dioxide

Chlorine dioxide has been generally recognized as a treatment for tastes caused by industrial wastes, such as phenols. However, chlorine dioxide can be used in the treatment of any taste and odour that is treatable by an oxidizing compound. Provisions should be made for proper storing and handling of the sodium chlorite, so as to eliminate any danger of explosion.

3.3.7.4. Powdered Activated Carbon

1. Powdered activated carbon should be added as early as possible in the treatment process to provide maximum contact time. Flexibility to allow the addition of carbon at several points is preferred. Activated carbon should not be applied near the point of chlorine or other oxidant application.

2. The carbon can be added as a pre-mixed slurry or by means of a dry-feed machine as long as the carbon is properly wetted.

3. Continuous agitation or re-suspension equipment is necessary to keep the carbon from depositing in the slurry storage tank.

4. Provision shall be made for adequate dust control.

5. The required rate of feed of carbon in a water treatment plant depends upon the tastes and/or odours involved, but provision should be made for adding from 0.l mg/L to at least 40 mg/L.

6. Powdered activated carbon shall be handled as a potentially combustible material. It should be stored in a building or compartment as nearly fireproof as possible. Other chemicals should not be stored in the same compartment. A separate room should be provided for carbon feed installations. Carbon feeder rooms should be equipped with explosion-proof electrical outlets, lights and motors.

3.3.7.5. Granular Activated Carbon

See Section 3.3.2.1.6 for application within filters.

3.3.7.6. Copper Sulphate and Other Copper Compounds

Continuous or periodic treatment of water with copper compounds to kill algae or other growths shall be controlled to prevent copper in excess of 1.0 mg/L as copper in the plant effluent or distribution system. Care shall be taken to ensure an even distribution. Approval for the dosage proposed must be obtained from the DOEC. For continuous feeding, copper compounds should be added by machine or solution-feed equipment. In large reservoirs, the chemical can be added from a boat by dragging bags of the chemical, by chemical sprayers or by using dry-feed or solution-feed machines.

3.3.7.7. Aeration

See Section 3.3.4.

3.3.7.8. Potassium Permanganate

Application of potassium permanganate may be considered, providing the treatment shall be designed so that the products of the reaction are not visible in the finished water.

3.3.7.9. Ozone

Ozonation can be used as a means of taste and odour control. Adequate contact time must be provided to complete the chemical reactions involved. Ozone is generally more desirable for treating water with high threshold odours.

3.3.7.10. Other Methods for Taste and Odour Control

The decision to use any other methods of taste and odour control should be made only after careful laboratory tests and/or pilot plant tests, and in consultation with the DOEC.

3.3.8. Fluoridation

3.3.8.1. Approval of Fluoridation Program

Proposals for fluoridation shall be submitted to the DOEC for approval. The proposal shall include:

1. Written endorsement or resolution of the local medical and dental societies and the local health authorities;
2. Submission of an appropriate fluoridation bylaw by the local government describing the condition, timing and general regulations that will be applied;
3. Detailed specifications for the chemical feeding equipment to be used;
4. Detailed plans showing the location of the equipment, piping layout, manner of control, and point of fluoride application;
5. Statement of the chemical to be used with quantitative analysis;
6. Plans for storage of the chemical to be used;
7. Plans for dust control facilities; and
8. Appraisal and approval of the qualifications of the personnel who will make the analyses to control the application of fluorides.

3.3.8.2. Fluoride Compounds

Various compounds are available and include sodium fluoride, sodium silicofluoride, ammonium silicofluoride and hydro-fluosilicic acid. The fluoride compounds shall conform to the applicable AWWA standards.

3.3.8.3. Fluoride Storage Facilities

Fluoride chemicals should be isolated from other chemicals to prevent contamination. The fluoride chemicals should be stored in covered or unopened shipping containers, unless the chemical is transferred into an approved covered storage unit and away from acids. Storage units directly supplying feeders should have sufficient capacity for one day's dosage at the average daily demand. Chemicals must be stored in a reasonably dry space. Storage units for large quantities of hydro-fluorosilicic acid should be vented to the atmosphere.

3.3.8.4. Chemical Feeders

1. Chemical feeders should be selected to meet specific requirements and hydraulic conditions. The accuracy should be within 5% of the intended dosage. Scales, loss-of-weight recorders or liquid level indicators shall be provided for chemical feeds.

2. Where the rate of flow of the water being treated varies over short intervals of time, the feeder should dose in proportion to the flow. Scales or loss-of-mass recorders should be provided.

3. The floor surfaces surrounding the feeders should be smooth and impervious.

4. Fluoride compound shall not be added before lime-soda softening or ion exchange softening.

5. The point of application of fluorosilicic acid, if into a horizontal pipe, shall be in the lower half of the pipe.

6. A fluoride solution shall be applied by a positive displacement pump having a stroke rate not less than 20 strokes per minute.

7. A spring opposed diaphragm type anti-siphon device shall be provided for all fluoride feed lines and dilution water lines.

8. A device to measure the flow of water to be treated is required.

9. The dilution water pipe shall terminate at lease two pipe diameters above the solution tank.

10. Water used for sodium fluoride dissolution shall be softened if hardness exceeds 75 mg/L as calcium carbonate.

11. Fluoride solutions shall be injected at a point of continuous positive pressure or a suitable air gap provided.

12. The electric outlet used for the fluoride feed pump should have a non-standard receptacle and shall be interconnected with the well or service pump.

13. Saturators shall be of the upflow type and be provided with a meter and backflow protection on the makeup water line.

The following apply to dry chemical feeders:

1. Dry chemical feeders of either the volumetric or gravimetric type are acceptable.

2. Dry chemical feeders must be completely enclosed and precautions for dust prevention shall be taken.

3. Water should be fed to the solution pot so as to prevent back-siphonage into the water supply. A vacuum breaker or its equivalent should be provided to prevent the solution from being drained or siphoned into the water supply when the unit is shut down.

4. There should be no direct connection between any sewer and the drain of a solution pit.

5. Any booster pump used to force the solution into the water should be constructed of a material not subject to chemical attack (e.g. bronze).

3.3.8.5. Protective Equipment

The following shall be provided for each operator:

1. At least one pair of rubber gloves with long gauntlet;

2. A dust respirator of a type approved by the DOEC for toxic dusts;

3. An apron or other protective clothing;

4. Goggles or face mask; and

5. Other protective equipment must be provided as necessary.

3.3.8.6. Dust Control

1. Provision should be made for the disposal of empty bags, drums or barrels, by approved methods that will minimize exposure to fluoride dusts.

2. A metal wheelbarrow should be available for the temporary handling of punctured bags.

3. Provision should be made for the removal of fluoride dust from floors and equipment, either by wet mopping or a suitable type of vacuum cleaner. Floor drains should be provided at large installations to facilitate the hosing of floors.

3.3.8.6.1. Dry Conveyors

Provision shall be made for the transfer of dry fluoride compounds from shipping containers to storage bins or hoppers to minimize the amount of fluoride dust

3.3.8.6.2. Dust Control Procedures

Plans and specifications for fluoridation equipment and facilities will be reviewed from the standpoint of proposed dust-prevention procedures. Approval will be considered only when they conform to the following:

1. Vacuum pneumatic equipment for drawing powdered material from shipping containers to closed storage hoppers, with the exhaust air from the system effectively filtered and discharged to the exterior.

2. An exhaust fan, with a dust filter and suitable ducts, having capacity to provide a flow of entering air at a velocity of at least 50 m/min at the opening through which the compound is dumped into an otherwise closed hopper or bin from bags, drums or barrels. (The capacity of the fan, therefore, should be selected with due regard to the area of this opening).

3. An enclosure, forming an integral part of the chemical feeder, into which a bag or drum of a fluoride compound may be placed before the container is emptied, so that the emptying process takes place within the enclosure.

4. A drum equipped with a tight-fitting adapter that will allow the drum to be inverted and connected tightly to a mating adapter on top of the hopper, said adapter incorporating a slide gate or other dust-tight means for opening the drum to the hopper after tight connection has been made thereto.

5. A small-capacity dry-feeder fitted with a covered hopper, where only a small quantity of a powdered crystalline or granular form a fluoride compound will be transferred into the hopper at one time, by means of a small hand utensil. (The use of the crystalline or granular form of these compounds is preferred, when so handled).

6. A solution tank containing water, into which a powdered, crystalline, granular, pellet or tablet form of the material is to be placed to form a solution.

7. Any other approved equipment which may be developed for use with a specific type of shipping container or device, which will permit the transfer of the fluoride compound to an enclosed chemical feeder without the release of dust.

3.3.8.6.3. Approval

Complete details of the special provisions for dust prevention shall be included with the submission to the DOEC for the approval of any fluoridation facility.

3.3.8.7. Testing Equipment

Equipment shall be provided for the routine testing of the fluoride ion concentration in the raw and treated water. The equipment shall meet the requirements of the DOEC.

3.3.9. Chemical Application

3.3.9.1. Choice of Chemicals

The choice of chemicals should be determined by on-site or pilot plant tests (e.g. choice of coagulants and coagulant aids). The cost and availability of the chemical should be of prime consideration.

3.3.9.2. Chemical Feed Devices

1. The arrangement of the chemical feed machines and choice of facilities should ensure uniform and continuous treatment.

2. Dry feed, volumetric or gravimetric, or solution feed types are satisfactory and should be provided with a minimum of one standby unit.

3. Feed machines should be equipped with alarm devices to warn operators of failures, and should be capable of ready adjustment to variations in raw water flow.

4. The delivery capacity should be sufficient to supply the required chemical dosage to treat raw water effectively when flowing at the maximum design rate. Devices for recording feed rates are desirable.

5. All feed machines should be located as close as feasible to the point of application of chemicals. Long solution lines or lines encased in floors or walkways should be avoided.

6. It is desirable that chemical solutions, especially suspensions of lime, be conducted in short open flumes. Provision should be made for adequate velocity in the lines to keep the chemical in suspension.

7. Chemical storage rooms should be separate from the feed machine room, and should provide for at least one month's storage of chemicals. Consideration should be given to the dust problem inherent to the application of dry chemicals.

3.3.9.3. Mixing Chamber

1. Retention periods for any type of mixing chamber should be sufficient to ensure thorough mixing, but not long enough to permit formation and settling of the floc.

2. Mechanical mixing devices should be adjustable with respect to mixing rate to account for variations in raw water quality and flow. Mixing devices may be mechanical units, baffled basins, hydraulic jump, aerating devices or other approved types. Selection would depend upon each specific requirement.

3.3.10. Pre-engineered Water Treatment Plants

Pre-engineered water treatment plants are normally modular process units, which are pre-designed for specific process applications and flow rates, and purchased as a package. Multiple units may be installed in parallel to accommodate larger flows.

Pre-engineered treatment plants have numerous applications but are especially applicable for small systems where conventional treatment may not be cost effective. As with any design, the proposed treatment must fit the situation and ensure a continuous supply of safe drinking water for water consumers. The DOEC may accept proposals for pre-engineered water treatment plants on a case by base basis where they have been demonstrated to be effective in treating the source water being used.

Factors to be considered include:

1. Raw water quality characteristics under normal and worst case conditions; seasonal fluctuations must be evaluated and considered in the design;

2. Demonstration of treatment effectiveness under all raw water conditions and systems flow demands; this demonstration may be on-site pilot or full scale testing or testing off-site where the source water is of similar quality. On-site testing is required at sites having questionable water quality or applicability of the treatment process. The proposed demonstration project must be approved by the DOEC prior to commencement;

3. Sophistication of equipment; the reliability and experience record of the proposed treatment equipment and controls must be evaluated;

4. Unit process flexibility which allows for optimization of treatment;

5. Operational oversight that is necessary; at surface water sources, full-time operators are necessary except where the DOEC has approved an automation plan.

6. Third party certification or approval such as National Sanitation Foundation (NSF) for:
   a. Treatment equipment; and

   b. Materials that will be in contact with the water.

7. Suitable pretreatment based on raw water quality and the pilot study or other demonstration of treatment effectiveness;

8. Factory testing of controls and process equipment prior to shipment;

9. Automated troubleshooting capability built into the control system;

10. Start-up and follow-up training and troubleshooting to be provided by the manufacturer or contractor;

11. Operation and maintenance manual; this manual must provide a description of the treatment,

control and pumping equipment, necessary maintenance and schedule, and a troubleshooting guide for typical problems;

12. On-site and contractual laboratory capability; the on-site testing must include all required continuous and daily testing as specified by the DOEC. Contract testing may be considered for other parameters;

13. Manufacturers warranty and replacement guarantee. Appropriate safeguards for the water supplier must be included in contract documents. The DOEC may consider interim or conditional project approvals for innovative technology where there is sufficient demonstration of treatment effectiveness and contract provisions to protect the water supplier should the treatment not perform as claimed; and

14. Water supplier revenue and budget for continuing operations, maintenance and equipment replacement in the future.

3.3.11. Automated/Unattended Operation of Surface Water Treatment Plants

Recent advances in computer technology, equipment controls and SCADA systems have brought automated and off-site operation of surface water treatment plants into the realm of feasibility. Coincidentally, this comes at a time when renewed concern for microbiological contamination is driving optimization of surface water treatment plant facilities and operations and finished water treatment goals are being lowered to levels of <0.1 NTU turbidity and <20 total particle counts per millilitre.

The DOEC encourages any measure, including automation, which assists operators in improving plant operations and surveillance functions.

Automation of surface water treatment facilities to allow unattended operation and off-site control presents a number of management and technological challenges which must be overcome before an Environmental Permit can be considered. Each fact of the plant facilities and operations must be fully evaluated to determine what on-line monitoring is appropriate, what alarm capabilities must be incorporated into the design, and what staffing is necessary. Consideration must be given to the consequences and operational response to treatment challenges, equipment failure and loss of communications or power.

A Technical Report shall be developed as the first step in the process leading to design of the automation system. The Technical Report to be submitted to the DOEC must cover all aspects of the treatment plant and automation system including the following information/criteria:

1. Identify all critical features in the pumping and treatment facilities that will be electronically monitored, have alarms and can be operated automatically or off-site via the control system. Include a description of automatic plant shutdown controls with alarms and conditions, which would trigger shutdowns. Dual or secondary alarms may be necessary for certain critical functions.

2. Automated monitoring of all critical functions with major and minor alarm features must be

provided. Automated plant shutdown is required on all major alarms. Automated start-up of the plant is prohibited after shutdown due to a major alarm. The control system must have response and adjustment capability on all minor alarms. Built-in control system challenge test capability must be provided to verify operational status of major and minor alarms.

3. The plant control system must have the capability for manual operation of all treatment plant equipment and process functions.

4. A plant flow diagram, which shows the location of all, critical features, alarms and automated controls, to be provided.

5. Description of off-site controls station(s) that allow observation of plant operations, receiving alarms and having the ability to adjust and control operation of equipment and the treatment process.

6. An operator shall be on "standby duty" status at all times with remote operational capability and located within a reasonable response time of the treatment plant.

7. An operator shall do an on-site check at least once per day to verify proper operation and plant security.

8. Description of operator staffing and training, planned or completed, in both process control and the automation system.

9. Operations manual, which gives operators step by step procedures for understanding and using the automated control system under all water quality conditions. Emergency operations during power or communications failures or other emergencies must be included.

10. A plan for a 6 month, or more, demonstration period to prove the reliability of procedures, equipment and surveillance system. An operator shall be on-duty during the demonstration period. The final plan must identify and address any problems and alarms that occurred during the demonstration period. Challenge testing of each critical component of the overall system must be included as part of the demonstration project.

11. Schedule for maintenance of equipment and critical parts replacement.

12. Sufficient finished water storage shall be provided to meet system demands and Ct requirements whenever normal treatment production is interrupted as the result of automation system failure or plant shutdown.

13. Sufficient staffing must be provided to carry out daily on-site evaluations, operational functions and needed maintenance and calibration of all critical treatment components and monitoring equipment to ensure reliability of operations.

14. Plant staff must perform, as a minimum, weekly checks on the communication and control system to ensure reliability of operations. Challenge testing of such equipment should be

part of normal maintenance routines.

15. Provisions must be made to ensure security of the treatment facilities at all times. Incorporation of appropriate intrusion alarms must be provided which are effectively communicated to the operator in charge.

3.3.12. Waste Handling and Disposal

Provisions must be made for the proper disposal of water treatment plant waste, such as sanitary, laboratory, clarification sludge, softening sludge, iron sludge, filter backwash water, and brines. All waste discharges shall require the approval of the DOEC as per provisions of the *Environmental Protection Act*. The requirements outlined herein must, therefore, be considered minimum requirements as other regulatory agencies may have more stringent requirements.

In locating waste disposal facilities due consideration should be given to preventing potential contamination of the water supply.

Alternative methods of water treatment and chemical use should be considered as a means of reducing waste handling and disposal problems.

3.3.12.1. Specific Wastes

3.3.12.1.1. Sanitary Waste

The sanitary waste from water treatment plants, pumping stations, and other water works installations must receive treatment. Waste from these facilities must be discharged directly to a sanitary sewer system, or to an adequate on-site waste treatment facility approved by the DOEC.

3.3.12.1.2. Brine Waste

Waste from ion exchange plants, demineralization plants, or other plants, which produce a brine, may be disposed of by controlled discharge to a stream if adequate dilution is available. Surface water quality requirements of the DOEC will control the rate of discharge. Except when discharging to large waterways, a holding tank of sufficient size should be provided to allow the brine to be discharged over a twenty-four hour period. Where discharging to sanitary sewer, a holding tank may be required to prevent overloading of the sewer and/or interference with the waste treatment process. The effect of brine discharge to wastewater treatment ponds may depend on the rate of evaporation from the ponds.

3.3.12.1.3. Sludge

Sludge from plants using lime to soften water varies in quantity and in chemical characteristics depending on the softening process and the chemical characteristics of the water being softened. Recent studies show that the quantity of sludge produced is much larger than indicated by stoic-metric calculations. Methods of treatment and disposal are as follows:

1. Wastewater Treatment Ponds (Lagoons):

a) Temporary wastewater treatment ponds which must be cleaned periodically should be designed on the basis of 2833 m^2 per 3785 m^3 per day per 100 mg/l of hardness removed based on a usable pond depth of 1.5m. This should provide about 2.5 years storage. At least two but preferably more wastewater treatment ponds must be provided in order to give flexibility in operation. An acceptable means of final sludge disposal must be provided. Provisions must be made for convenient cleaning.

b) Permanent wastewater treatment ponds should have a volume of at least four times that for temporary wastewater treatment ponds.

c) The design of both temporary and permanent wastewater treatment ponds should provide for:
   i. Location free from flooding;

   ii. When necessary, dikes, deflecting gutters or other means of diverting surface water so that it does not flow into the wastewater treatment ponds;

   iii. A minimum usable depth of 1.5 m;

   iv. Adequate freeboard of at least 600 mm;

   v. Adjustable decanting device;

   vi. Effluent sampling point;

   vii. Adequate safety provisions; and

   viii. Parallel operation.

2. The application of liquid lime sludge to farmland should be considered as a method of ultimate disposal. Prior to land application, a chemical analysis of the sludge including calcium and heavy metals shall be conducted. Approval from the DOEC and other concerned agencies must be obtained. When this method is selected, the following provisions shall be made:
   a) Transport of sludge by vehicle or pipeline shall incorporate a plan or design which prevents spillage or leakage during transport;

   b) Interim storage areas at the application site shall be kept to a minimum and facilities shall be provided to prevent wash off of sludge or flooding;

   c) Sludge shall not be applied at times when wash off of sludge from the land could be expected;

   d) Sludge shall not be applied to sloping land where wash off could be expected unless provisions are made, for suitable land, to immediately incorporate the sludge into the soil;

e) Trace metals loading shall be limited to prevent significant increases in trace metals in the food chain, phytotoxicity or water pollution; and

f) Each area of land to receive lime sludge shall be considered individually and a determination made as to the amount of sludge needed to raise soil pH to the optimum for the crop to be grown.

3. Discharge of lime sludge to sanitary sewers is not permitted.

4. Mixing of lime sludge with activated sludge waste may be considered as a means of co-disposal.

5. Mechanical dewatering of sludge may be considered. Pilot studies on a particular plant waste are required.

6. Calcination of sludge may be considered. Pilot studies on a particular plant waste are required.

7. Lime sludge drying beds are not recommended.

3.3.12.1.4. Red Water Waste
Waste filter wash water from iron and manganese removal plants can be disposed of as per Section 3.3.12.2.

3.3.12.1.5. Filter Wash Water
Waste filter wash water from surface water treatment or lime softening plants should have suspended solids reduced to a level acceptable to the DOEC before being discharged.

Under special circumstances, a plant may be permitted to construct a holding or sludge concentration tank. The tank should be of such a size that it would contain the anticipated volume of such wastewater produced by the plant when operating at design capacity. A plant that has two filters should have a tank that will contain the total waste wash water from both filters calculated by using a 15-minute wash at 815 L/min/m^2. In plants with more filters, the size of the tank will depend on the anticipated hours of operation.

3.3.12.2. Waste Disposal

3.3.12.2.1. Sand Filters
Sand filters can be used in the disposal of sludge, and should have the following features:
1. Total filter area shall be sufficient to adequately dewater applied solids. Unless the filter is small enough to be cleaned and returned to service in one day, two or more cells are required.

2. The filter shall have sufficient capacity to contain, above the level of the sand, the entire volume of wash water produced by washing all of the production filters in the plant, unless the production filters are washed on a rotating schedule and the flow through the production filters is regulated by the rate of flow controllers. In this case, sufficient volume must be provided to properly dispose of the wash water involved.

3. Sufficient filter surface area should be provided so that, during any one-filtration cycle, no more than 60 cm of backwash water will accumulate over the sand surface.

4. The filter shall not be subject to flooding by surface runoff or floodwaters. Finished grade elevation shall be established to facilitate maintenance, cleaning and removal of surface sand as required. Flashboards or other non-watertight devices shall not be used in the construction of filter sidewalls.

5. The filter media should consist of a minimum of 300 mm of sand, 75 to 100 mm of supporting small gravel or torpedo sand, and 225 mm of gravel in graded layers. All sand and gravel should be washed to remove fines.

6. Filter sand should have an effective size of 0.3 to 0.5 mm and a uniformity coefficient not to exceed 3.5. The use of larger sized sands shall be justified by the designing engineer, to the satisfaction of the DOEC.

7. The filter should be provided with an adequate under-drainage collection system to permit satisfactory discharge of filtrate.

8. Provision shall be made for the sampling of the filter effluent.

9. Overflow devices from the filters shall not be permitted.

10. Where freezing is a problem, provisions should be made for covering the filters during the winter months.

11. Filters shall comply with the common wall provisions contained in Sections 3.6.8.3 and 3.7.7, which pertain to the possibility of contaminating treated water with unsafe water. The DOEC must be contacted for approval of any arrangement where a separate structure is not provided.

3.3.12.2.2. Wastewater Treatment Ponds (Lagoons)

Wastewater treatment ponds shall have the following features:

1. Be designed with volume 10 times the total quantity of wash water discharged during any 24-hour period;

2. A minimum usable depth of 900 mm;

3. Length four times width, and the width at least three times the depth, as measured at the operating water level;

4. Outlet to be at the end opposite the inlet;

5. A weir overflow device at the outlet end with weir length equal to or greater than depth; and

6. Velocity to be dissipated at the inlet end.

3.3.12.2.3. Recycling Waste Filtrates
Recycling of supernatant or filtrate from waste treatment facilities to the head end of the plant shall not be allowed except as approved by the DOEC.

3.4. Chemical Application

Chemicals shall be applied to the water at such points and by such means as to:
1. Assure maximum efficiency of treatment;

2. Assure maximum safety to the consumer;

3. Provide maximum safety to the operators;

4. Assure satisfactory mixing of the chemicals with the water;

5. Provided maximum flexibility of operation through various points of application, when appropriate; and

6. Prevent backflow or back-siphonage between multiple points of feed through common manifolds.

3.4.1. Approval
No chemicals shall be applied to treat drinking waters unless specifically permitted by the DOEC.

Plans and specifications shall be submitted for review and approval, as provided for in Section 2, and shall include:
1. Descriptions of feed equipment, including maximum and minimum feed ranges;

2. Location of feeders, piping layout and points of application;

3. Storage and handling facilities;

4. Specifications for chemicals to be used;

5. Operating and control procedures including proposed application rates; and

6. Descriptions of testing equipment and procedures.

3.4.2. General Equipment Design

General equipment design shall be such that:

1. Feeders will be able to supply, at all times, the necessary amounts of chemicals at an accurate rate, throughout the range of feed;

2. Chemical-contact materials and surfaces are resistant to the aggressiveness of the chemical solution;

3. Corrosive chemicals are introduced in such a manner as to minimize potential for corrosion;

4. Chemicals that are incompatible are not stored or handled together;

5. All chemicals are conducted from the feeder to the point of application in separate conduits;

6. Chemical feeders are as near as practical to the feed point;

7. Chemical feeders and pumps should operate as per the manufacturer's recommendations; and

8. Chemicals are fed by gravity where practical.

3.4.3. Facility Design

3.4.3.1 Number of Feeders

1. Where chemical feed is necessary for the protection of the supply, such as chlorination, coagulation or other essential processes:
   a) A minimum of two feeders shall be provided;

   b) The standby unit or a combination of units of sufficient capacity should be available to replace the largest unit during shut-downs; and

   c) Where a booster pump is required, duplicate equipment shall be provided and, when necessary, standby power.

2. A separate feeder shall be used for each chemical applied.

3. Spare parts shall be available for all feeders to replace parts, which are subject to wear and damage.

3.4.3.2. Control of Facility

1. Feeders may be manually or automatically controlled, with automatic controls being designed so as to allow override by manual controls.

2. At automatically operated facilities, chemical feeders shall be electrically interconnected with the well or service pump and should be provided a non-standard electrical receptacle.

3. Chemical feed rates shall be proportional to flow.

4. A means to measure water flow must be provided in order to determine chemical feed rates.
5. Provisions shall be made for measuring the quantities of chemicals used.

6. Weighing scales:
   a) Shall be provided for weighing cylinders at all plants utilizing chlorine gas;

   b) May be required for fluoride solution feed;

   c) Should be provided for volumetric dry chemical feeders; and

   d) Shall be capable of providing reasonable precision in relation to average daily dose.

7. Where conditions warrant, for example with rapidly fluctuating intake turbidity, coagulant and coagulant aid addition may be made according to turbidity, streaming current or other sensed parameter.

3.4.3.3. Dry Chemical Feeders

Dry chemical feeders shall:
1. Measure chemicals volumetrically or gravimetrically;

2. Provide adequate solution water and agitation of the chemical in the solution tank;

3. Provide gravity feed from solution pots; and

4. Completely enclose chemicals to prevent emission of dust to the operating room.

3.4.3.4. Positive Displacement Solution Pumps

Positive displacement type solution feed pumps should be used to feed liquid chemicals, but shall not be used to feed chemical slurries. Pumps must be capable of operating at the required maximum rate against the maximum head conditions found at the point of injection.

3.4.3.5. Liquid Chemical Feeders – Siphon Control

Liquid chemical feeders shall be such that chemical solutions cannot be siphoned into the water supply, by:
1. Assuring discharge at a point of positive pressure;

2. Providing vacuum relief;

3. Providing a suitable air gap; or

4. Providing other suitable means or combinations as necessary.

3.4.3.6. Cross-connection Control

Cross-connection control must be provided to assure that:

1. The service water lines discharging to solution tanks shall be properly protected from backflow as required by the DOEC (see Section 4.2.5.7);

2. Liquid chemical solutions cannot be siphoned through solution feeders into the water supply as required in Section 3.4.3.5; and

3. No direct connection exists between any sewer and a drain or overflow from the feeder, solution chamber or tank by providing that all drains terminate at least 150 mm or two pipe diameters, whichever is greater, above the overflow rim of a receiving sump, conduit or waste receptacle.

3.4.3.7. Chemical Feed Equipment Location

Chemical feed equipment shall be:

1. Readily accessible for servicing, repair, and observation of operation;

2. Located in a separate room where required to reduce hazards and dust problems;

3. Conveniently located near points of application to minimize length of feed lines; and

4. Located such that the flow to the rapid mix is by gravity.

3.4.3.8. In-plant Water Supply

In-plant water supply shall be:

1. Ample in quantity and adequate in pressure;

2. Provided with means for measurement when preparing specific solution concentrations by dilution;

3. Properly treated for hardness, when necessary;

4. Properly protected against backflow, and

5. Obtained from a location sufficiently downstream of any chemical feed point to assure adequate mixing.

3.4.3.9. Storage of Chemicals

1. Space should be provided for:
   a) At least 30 days of chemical supply;

   b) Convenient and efficient handling of chemicals;

   c) Dry storage conditions; and

d) A minimum storage volume of 1½ truck loads where purchase is by truckload lots.

2. Storage tanks and pipelines for liquid chemicals shall be specified for use with individual chemicals and not used for different chemicals.

3. Chemicals shall be stored in covered or unopened shipping containers, unless the chemical is transferred into an approved storage unit.

4. Liquid chemical storage tanks must:

   a) Have a liquid level indicator; and

   b) Have an overflow and a receiving basin capable of receiving accidental spill or overflows without uncontrolled discharge.

3.4.3.10. Solution Tanks

1. A means, which is consistent with the nature of the chemical solution, shall be provided in a solution tank to maintain a uniform strength of solution. Continuous agitation shall be provided to maintain slurries in suspension.

2. Two solution tanks of adequate volume may be required for a chemical to assure continuity of supply while servicing a solution tank.

3. Means shall be provided to measure the liquid level in the tank.

4. Chemical solutions shall be kept covered. Large tanks with access openings shall have such openings curbed and fitted with overhanging covers.

5. Subsurface locations for solution tanks shall:
   a) Be free from sources of possible contamination; and

   b) Assure positive drainage for groundwater, accumulated water, chemical spills and overflows

6. Overflow pipes, when provided, should:
   a) Be turned downward, with the end screened;

   b) Have a free fall discharge; and

   c) Be located where noticeable.

7. Acid storage tanks must be vented to the outside atmosphere, but not through vents in common with day tanks.

8. Each tank shall be provided with a valved drain, protected against backflow in accordance with Sections 3.4.3.5 and 3.4.3.6.

9. Solution tanks shall be located and protective curbings provided so that chemicals from equipment failure, spillage or accidental drainage shall not enter the water in conduits, treatment or storage basins.

3.4.3.11. Day tanks

1. Day tanks shall be provided where bulk storage of liquid chemical is provided.

2. Day tanks shall meet all the requirements of Section 3.4.3.10.

3. Day tanks should hold no more than a 30-hour supply.

4. Day tanks shall be scale-mounted, or have a calibrated gauge painted or mounted on the side if liquid level can be observed in a gauge tube or through translucent sidewalls of the tank. In opaque tanks, a gauge rod extending above a reference point at the top of the tank, attached to a float may be used. The ratio of the area of the tank to its height must be such that unit readings are meaningful in relation to the total amount of chemical fed during a day.

5. Hand pumps may be provided for transfer from a carboy or drum. A tip rack may be used to permit withdrawal into a bucket from a spigot. Where motor-driven transfer pumps are provided, a liquid level limit switch and an over-flow from the day tank, must be provided.

6. A means, which is consistent with the nature of the chemical solution, shall be provided to maintain uniform strength of solution in a day tank. Continuous agitation shall be provided to maintain chemical slurries in suspension.

7. Tanks and tank refilling line entry points shall be clearly labelled with the name of the chemical contained.

3.4.3.12. Feed Lines

1. Should be as short as possible, and:
   a) Be of durable, corrosion-resistant material;

   b) Easily accessible throughout the entire length;

   c) Protected against freezing; and

   d) Readily cleanable.

2. Should slope upward from the chemical source to the feeder when conveying gases;

3. Shall be designed consistent with scale-forming or solids depositing properties of the water, chemical, solution or mixtures conveyed; and

4. Should be colour-coded.

3.4.3.13. Handling

1. Carts, elevators and other appropriate means shall be provided for lifting chemical containers to minimize excessive lifting by operators.

2. Provisions shall be made for disposing of empty bags, drums or barrels by an approved procedure, which will minimize exposure to dusts.

3. Provision must be made for the proper transfer of dry chemicals from shipping containers to storage bins or hoppers, in such a way as to minimize the quantity of dust, which may enter the room in which the equipment is installed. Control should be provided by use of:
   a) Vacuum pneumatic equipment or closed conveyor systems;

   b) Facilities for emptying shipping containers in special enclosures; and/or

   c) Exhaust fans and dust filters, which put the hoppers or bins under negative pressure.

4. Provision shall be made for measuring quantities of chemicals used to prepare feed solutions.

3.4.3.14. Housing

1. Floor surfaces shall be smooth and impervious, slip-proof and well drained with 2.5 %, minimum slope.

2. Vents from feeders, storage facilities and equipment exhaust shall discharge to the outside atmosphere above grade and remote from air intakes or exit doors.

3.4.4. Chemicals

3.4.4.1. Shipping Containers

Chemical shipping containers shall be fully labelled to include:
1. Chemical name, purity and concentration; and

2. Supplier name and address.

3.4.4.2. Specifications

Chemicals and water contact materials shall meet ANSI/AWWA quality standards and ANSI/NSF Standard 60 or 61 safety standards.

3.4.4.3. Assay

Provisions may be required for assay of chemicals delivered.

3.4.5. Operator Safety

3.4.5.1. Ventilation

Special provisions shall be made for ventilation of chemical feeder and storage rooms.

3.4.5.2. Protective Equipment

1. At least one pair of rubber gloves, a dust respirator of a type certified by Occupational Health and Safety for toxic dusts, an apron or other protective clothing and goggles or face mask shall be provided for each operator as required by the DOEC. A deluge shower and/or eye-washing device should be installed where strong acids and alkalis are used or stored.

2. A water holding tank that will allow water to come to room temperature must be installed in the water line feeding the deluge shower and eye-washing device. Other methods of water tempering will be considered on an individual basis.

3. Other protective equipment should be provided as necessary.

3.4.6. Specific Chemicals

3.4.6.1. Acids and Caustics

1. Acids and caustics shall be kept in closed corrosion-resistant shipping containers or storage units.

2. Acids and caustics shall not be handled in open vessels, but should be pumped in undiluted form from original containers through suitable hose, to the point of treatment or to a covered day tank.

3.4.6.2. Sodium Chlorite for Chlorine Dioxide Generation

Proposals for the storage and use of sodium chlorite must be approved by the DOEC prior to the preparation of final plans and specifications. Provisions shall be made for proper storage and handling of sodium chlorite to eliminate any danger of fire or explosion associated with its powerful oxidizing nature.

1. **Storage:**
   a) Chlorite (sodium chlorite) shall be stored by itself in a separate room and preferably shall be stored in an outside building detached from the water treatment facility. It must be stored away from organic materials because many materials will catch fire and burn violently when in contact with chlorite.

   b) The storage structures shall be constructed of non-combustible materials.

   c) If the storage structure must be located in an area where a fire may occur, water must be available to keep the sodium chlorite area cool enough to prevent heat induced explosive decomposition of the chlorite.

2. **Handling:**

a) Care should be taken to prevent spillage.

b) An emergency plan of operation should be available for the cleanup of any spillage.

c) Storage drums must be thoroughly flushed prior to recycling or disposal.

3. Feeders:

a) Positive displacement feeders shall be provided.

b) Piping for conveying sodium chlorite or chlorine dioxide solutions shall be Type 1 PVC, polyethylene or materials recommended by the manufacturer.

c) Chemical feeders may be installed in chlorine rooms if sufficient space is provided or facilities meeting the requirements of subsection 3.4.1 shall be provided.

d) Feed lines shall be installed in a manner to prevent formation of gas pockets and shall terminate at a point of positive pressure.

e) Check valves shall be provided to prevent the backflow of chlorine into the sodium chlorite line.

3.5. Pumping Facilities

Pumping facilities shall be designed to maintain the sanitary quality of pumped water. Subsurface pits or pump rooms and inaccessible installations should be avoided. No pumping stations should be subject to flooding.

Where raw water is drawn from surface water (streams, lakes, reservoirs, etc.) the design of the intake conduit, and suction wall should receive special attention to prevent clogging caused by trash, silt settling, or ice formation.

3.5.1. Location

The pumping station shall be so located that the proposed site will meet the requirements for sanitary protection of water quality, hydraulics of the system and protection against interruption of service by fire, flood or any other hazard.

The station shall be:

1. Elevated to a minimum of 0.9 m above the 100 year flood elevation, or 0.9 m above the highest recorded flood elevation, whichever is higher, or protected to such elevations;

2. Readily accessible at all times unless permitted to be out of service for the period of inaccessibility;

3. Graded around the station so as to lead surface drainage away from the station; and

4. Protected to prevent vandalism and entrance by unauthorized persons or animals.

3.5.2. Pumping Stations

Both raw and finished water-pumping stations shall:

1. Have adequate space for the installation of additional units if needed, and for the safe servicing of all equipment;

2. Be of durable construction, fire and weather resistant and with outward-opening doors;

3. Have floor elevation of at least 150 mm above finished grade, and the underground structure waterproofed;

4. Have all floors drained in such a manner that the quality of the potable water will not be endangered. All floors should have a slope of at least 2.5% to a suitable drain; and

5. Provide a suitable outlet for drainage from pump glands without discharging onto the floor.

3.5.2.1. Suction Well

Suction wells shall:

1. Be watertight;

2. Have floors sloped to permit removal of water and entrained solids;

3. Be covered or otherwise protected against contamination; and

4. Have two pumping compartments or other means to allow the suction well to be taken out of service for inspection, maintenance or repair.

3.5.2.2. Equipment Servicing

Pump stations should be provided with:

1. Crane-ways, hoist beams, eyebolts, or other adequate facilities for servicing or removal of pumps, motors or other heavy equipment;

2. Openings in floors, roofs or wherever else needed for removal of heavy or bulky equipment; and

3. A convenient tool board, or other facilities as needed, for proper maintenance of the equipment.

3.5.2.3. Stairways and Ladders

Stairs are preferred in areas were there is frequent traffic or where supplies are transported by hand. They shall meet the requirements of the Department of Government Services, Occupational Health and Safety Division.

Stairways or ladders shall:

1. Be provided between all floors, and in pits or compartments which must be entered; and

2. Have handrails on both sides, and treads of non-slip material.

3.5.2.4. Heating

Provisions should be made for adequate heating for:

1. The comfort of the operator; and

2. The safe and efficient operation of the equipment.

In pump houses not occupied by personnel, only enough heat need be provided to prevent freezing of equipment or treatment process.

3.5.2.5. Ventilation

Ventilation should conform to existing local and/or provincial codes. Adequate ventilation should be provided for all pumping stations. Forced ventilation of at least six changes of air per hour shall be provided for:

1. All confined rooms, compartments, pits and other enclosures below ground floor; and

2. Any area where unsafe atmosphere may develop or where excessive heat may be built up.

3.5.2.6. Dehumidification

In areas where excess moisture could cause hazards to safety or damage to equipment, means for dehumidification should be provided.

3.5.2.7. Lighting

Pump stations should be adequately lighted throughout. All electrical work shall conform to the requirements of the Canadian Electrical Code and related agencies and to the relevant Provincial codes.

3.5.2.8. Sanitary and Other Conveniences

Except in the cases of small automatic stations or where such facilities are otherwise available, all pumping stations should be provided with potable water, lavatory and toilet facilities. Plumbing must be so installed as to prevent contamination of a public water supply. Wastes should be discharged to a disposal system approved by the DOEC.

3.5.3. Pumps

At least two pumping units should be provided. With any pump out of service, the remaining pump or pumps shall be capable of providing the maximum daily pumping demand of the system. The pumping units shall:

1. Have ample capacity to supply the peak demand against the required distribution system pressure without dangerous overloading;

2. Be driven by ~~a~~ prime movers able to meet the maximum horsepower condition of the pumps;

3. Have spare parts and tools readily available; and

4. Be served by control equipment that has proper heater and overload protection for air temperature encountered.

3.5.3.1. Suction Lift

Suction lift shall be:

1. Avoided, if possible, and

2. Within allowable limits, preferably less than 4.5 m.

Where suction lift is necessary, self-priming pumps shall be provided.

3.5.3.2. Priming

Prime water must not be of lesser sanitary quality than that of the water being pumped. Means should be provided to prevent backpressure or back-siphonage backflow. When an air-operated ejector is used, the screened intake shall draw clean air from a point at least 3 m above the ground or other source of possible contamination, unless the air is filtered by an apparatus approved by the DOEC. Vacuum priming may be used.

3.5.4. Booster Pumps

Booster pumps shall be located or controlled so that:

1. They will not produce negative pressure in their suction lines;

2. The intake pressure should be at least 140 kPa when the pump is in normal operation;

3. The automatic cut-off or low pressure controller shall maintain at least 70 kPa in the suction line under all operating conditions;

4. Automatic or remote control devices shall have a range between the start and cut-off pressure which will prevent excessive cycling; and

5. A bypass is available.

3.5.4.1. Duplicate Pumps

Each booster pumping station should contain not less than two pumps with capacities such that peak demand can be satisfied with the largest pump out of service.

3.5.4.2. Metering

All booster-pumping stations should contain a totalizer meter.

3.5.4.3. Inline Booster Pumps

In addition to the other requirements of this section, inline booster pumps should be accessible for servicing and repairs.

3.5.4.4. Individual Home Booster Pumps

Individual home booster pumps should not be considered or required for any individual service from the public water supply main.

3.5.5. Automatic and Remote Controlled Pumping Stations

All automatic pumping stations shall be provided with automatic signalling apparatus, which will report when the station is out of service. All remote controlled stations shall be electrically operated and controlled and have signalling apparatus of proven performance. Installation of electrical equipment shall conform to the applicable Canadian and Provincial electrical codes.

3.5.6. Appurtenances

3.5.6.1. Valves

Pumps shall be adequately valved to permit satisfactory operation, maintenance and repair of the equipment. If foot valves are necessary, they should have a net valve area of at least 2.5 times the area of the suction pipe and they should be screened. Each pump shall have a positive-acting check valve on the discharge side between the pump and the shut-off valve. When the pumps are operating with positive suction pressure shut off valves should be also located in the suction lines.

3.5.6.2. Piping

In general piping shall:

1. Be designed so that the friction losses will be minimized;
2. Not be subject to contamination;
3. Have watertight joints;
4. Be protected against surge or water hammer and provided with suitable restraints where necessary; and
5. Be such that each pump has an individual suction line or that the lines shall be so manifolded that they will insure similar hydraulic and operating conditions.

3.5.6.3. Gauges and Meters

Each pump shall have:

1. A standard pressure gauge on its discharge line;
2. A compound gauge on its suction line;

3. Recording gauges in the larger stations; and

4. A means for measuring the discharge.

The station should have indicating, totalizing, and recording metering of the total water pumped.

3.5.6.4. Water Seals

Water seals shall not be supplied with water of a lesser sanitary quality than that of the water being pumped. Where pumps are sealed with potable water and are pumping water of lesser sanitary quality the seal shall:

1. Be provided with either an approved reduced pressure principle backflow preventer or a break tank open to atmospheric pressure,

2. Where a break tank is provided, have an air gap of at least 150 mm or two times the pipe diameter, whichever is greater, between the feed line and the flood rim of the tank.

3.5.6.5. Controls

Pumps, their prime movers and accessories, shall be controlled in such a manner that they will operate at rated capacity without dangerous overload. Where two or more pumps are installed, provision should be made for alternation. Provision shall be made to prevent energizing the motor in the event of a backspin cycle. Electrical controls should be located above grade. Equipment shall be provided or other arrangements made to prevent surge pressures from activating controls, which switch on pumps or activate other equipment outside the normal design cycle of operation.

3.5.6.6. Standby Power

To ensure continuous service when the primary power has been interrupted, a power supply shall be provided from an auxiliary source. If onsite generators or engines provide standby power, the fuel storage and fuel line must be designed to protect the water supply from contamination, and shall be in compliance with the *Storage and Handling of Gasoline and Associated Products Regulations, 2003*. As well, approval from the Department of Government Services will be required.

3.5.6.7. Water Pre-lubrication

When automatic pre-lubrication of pump bearings is necessary and an auxiliary direct drive power supply is provided, the pre-lubrication line should be provided with a valved bypass around the automatic control so that the bearings can, if necessary, be lubricated manually before the pump is started or the pre-lubrication controls shall be wired to the auxiliary power supply.

3.6. Finished Water Storage

Water storage is essential for meeting all of the domestic, public, industrial, commercial and fire-flow demands of almost all public water systems. This section addresses the requirements of treated water storage.

3.6.1. Definitions

Age of Treated Water **–** The age of treated water is measured as the time from when disinfection took place.

Detention Time – Detention time (sometime known as retention time or residence time) is defined as the period during which the treated water remains in storage prior to entering the distribution system. This may not be a fixed period and is dependent on utilization of the treated water and mixing of the treated water in storage. There could also be significant detention time within the distribution system prior to water reaching the first customer.

Elevated Tank – Elevated Tanks generally consist of a water tank supported by a steel or concrete tower that does not form part of the storage volume. In general, an elevated tank supplies peak balancing flows. See Figure 3.1a.

Standpipe – A standpipe is a tank that is located on the ground surface and has a greater height than diameter. In most installations water in the upper portion of the tank is used for peak flow balancing (equalization), the remaining volume is for fire flow and emergency storage. See Figure 3.1b.

Reservoir – A treated water reservoir is a storage facility where the width/diameter is typically greater than the height and usually applies to large storage facilities.

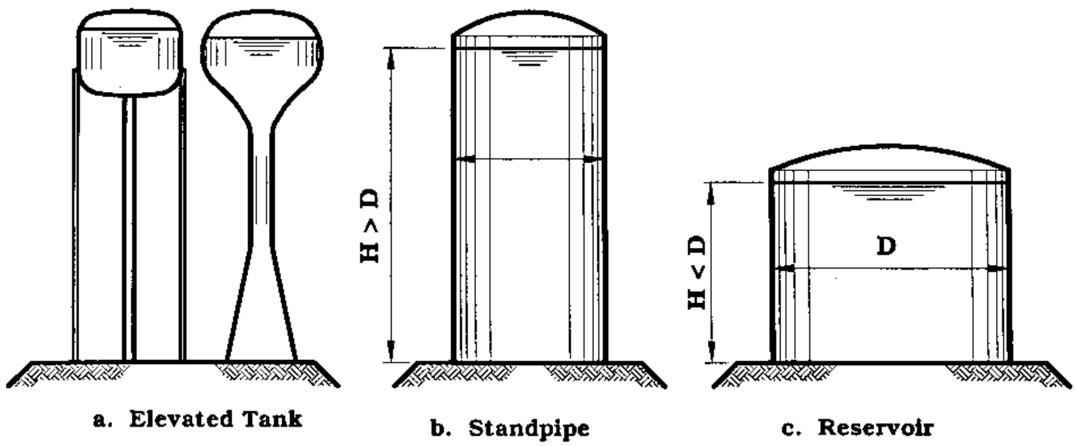
Above Ground Reservoir **–** An above ground reservoir is a water storage structure that is primarily above ground. See Figure 3.1c.

In-Ground Reservoir – An in-ground reservoir is a water storage structure that is partially below the nominal surface of the ground. A typical construction has the reservoir located 50% above and 50% below ground.

3.6.2. Hydropneumatic Systems

Hydropneumatic tanks are partly filled with water and partly filled with air. They are generally steel pressure tanks, with a flexible membrane that separates the air and the water. Air is compressed in the upper part of the tank and is used to maintain water pressure in the distribution system when demand exceeds the pump capacity. It also reduces on-off cycling of pumps.

Figure 3.1
Above Ground Storage



3.6.3. Materials of Construction

3.6.3.1. Standards and Materials Selection

Storage facilities, including pipes, fittings and valves, should conform to the latest standards issued by the CSA or AWWA, and be acceptable to the DOEC. In the absence of such standards, materials meeting applicable Product Standards and acceptable to the DOEC may be selected. Special attention should be given to selecting pipe materials that will protect against internal and external pipe corrosion. All products should comply with CSA/ANSI standards. Any material that comes in contact with drinking water must comply with NSF Standard 61.

Other materials of construction are acceptable when properly designed to meet the requirements of treated water storage including concrete.

3.6.3.2. Steel Construction

Steel structures should follow the current AWWA standards concerning steel tanks, standpipes, reservoirs, and elevated tanks wherever they are applicable. Painted welded steel and pre-finished bolted steel tanks are options for treated water storage tanks.

3.6.3.3. Concrete Construction

Concrete structures should follow the current AWWA standards concerning concrete tanks, standpipes, reservoirs, and elevated tanks wherever they are applicable.

3.6.4. Design Criteria

The top water level and location of the storage structures will be determined by the hydraulic analysis undertaken for the design of the distribution system to result in acceptable service pressures throughout the existing and future service areas.

The materials and design used for treated water storage structures should provide stability and durability as well as protect the quality of the stored water. The following subsections outline criteria that should be considered when designing treated water storage facilities.

3.6.4.1. Demand Equalization (Peak Balancing Storage)

The demand for water normally changes throughout the day and night. If treated water is not available from storage, the wells and/or treatment plant must have sufficient capacity to meet the demand at peak flow. The capacity is not generally practical or economical. With adequate storage, water can be treated or supplied to the system at a relatively uniform rate over a 24-hour period with peak balancing flows at high demand periods during the day being supplied by water storage tanks.

3.6.4.2. System Operation (Convenience)

In some situations, storage is provided to allow a treatment plant to be operated for only one or two shifts, thereby reducing personnel costs. In this situation storage provides the water required for the periods of time when the plant shuts down.

3.6.4.3. Smoothing Pumping Requirements

The demand for water is continually changing in all water systems, depending on time of day, day of the week, weather conditions and many other factors. If there is no storage at all, the utility has to continually match the changing demand by selecting pumps of varying sizes. Frequent cycling of pumps causes increased wear on controls and motors. It also increases energy costs. Adequate elevated storage can minimize this effect by providing peak flow balancing capacity.

3.6.4.4. Reducing Power Costs

Storage allows for pumping costs to be reduced, by reducing start-ups, avoiding using large pumps at peak demands and also benefiting from off-peak rates offered by the electricity utility during the night.

3.6.4.5. Emergency Storage

During periods of power failure, mechanical or pipeline breakdown or maintenance when use of source water is prevented, there is a need for emergency storage.

3.6.4.6. Fire Storage

Fire demands may not occur very often, however, when it does occur, the rate of water use is usually much greater than for domestic peak demand. Also, the required fire storage volume can account for as much as 50% of total capacity of the reservoirs.

3.6.4.7. Pressure Surge Relief

When pumps are turned on and off and when valves are opened and closed, large pressure changes can occur throughout the distribution system, which can damage pipes and appurtenances. Water storage tanks provide some assistance in absorbing pressure surges.

3.6.4.8. Detention Time

The time that water stays in storage after disinfectants are added, but before the water is delivered to the first customer, can be counted towards the disinfectant contact time.

Supplemental chlorination may be required to maintain minimum chlorine residuals in water from water storage facilities that has insufficient residual chlorine.

A detailed design of the inlet, outlet and baffling is required where storage facilities are used as supplemental chlorination stations.

3.6.4.9. Blending of Water Sources

Some water systems use water from two or more sources, with each source having different water quality. The feasibility of the blending of sources should be investigated, as the chemical quality of blended water may affect the integrity of the distribution system.

3.6.5. Sizing of Water Storage Facilities

Storage facilities should have sufficient capacity, as determined from engineering studies, to meet the required domestic demands, and where fire protection is provided, fire flow demands. Emergency storage volumes should be provided to supply demands in the event of pipeline or equipment breakdowns or maintenance shutdowns. Excessive storage capacity should be avoided where water quality deterioration may occur.

The total water storage requirements for a given water supply system where the treatment plant is capable of satisfying only the maximum day demand may be calculated using the following equation:

$$S = A + B + C$$

Where: S = Total Storage requirement, m^3;
A = Fire Storage, m^3 (equal to required fire flow over required duration);
B = Peak Balanced Storage, m^3 (25% of maximum day demand); and
C = Emergency Storage, m^3 (25% of A + B).

Notes:

1. The above equation is for the calculation of the storage requirement for a system where the water treatment plant is capable of satisfying only the maximum day demand. For situations where the water treatment plant can supply more, the above storage requirements can be reduced accordingly.
2. The maximum day demand referred to in the foregoing equation should be calculated using the factors in Table 3.7, unless there is existing flow data available to support the use of different factors. Where existing data is available, the required storage should be calculated on the basis of an evaluation of the flow characteristics within the system.

3. Should the proponent have decided to provide a potable water supply and distribution system not capable of providing fire protection, the usable volume of storage to be provided should be 25% of design year maximum day plus 40% of the design year average day.

4. The designer should recognize that this formula for calculating treated water storage requirements must be supplemental with the plant water storage required for the operation of the water treatment facility (i.e. backwash and domestic use).

3.6.5.1. Fire Flow Storage Requirements

The level of fire protection is the responsibility of the municipality. Fire flow requirements, typically established by the appropriate Insurance Advisory Organization (IAO), should be satisfied where fire protection is provided. The level of storage may be further reduced if the water treatment plant is capable of supplying portions of the required fire-flow volumes.

3.6.5.2. Peak Balancing Storage Requirements

Peak balancing storage also known as operational storage is directly related to the amount of water necessary to meet peak demands. The intent of peak balancing storage is to make up the difference between the consumer’s peak demands and the system’s available supply. With peak balancing storage, system pressures are typically improved and stabilized. The value of the peak balancing storage is a function of the diurnal demand fluctuation in a community and is commonly estimated at 25% of the total maximum day demand.

3.6.5.3. Emergency Storage

This is the volume of water recommended to meet the demand during maintenance shut-downs or emergency situations, such as source of supply failures, watermain failures, electrical power outages, or natural disasters. The amount of emergency storage included with a particular water system is not set, but is typically based on an assessment of risk and desired degree of system dependability.

In considering emergency storage, it is acceptable to evaluate providing significantly reduced supplies during emergencies.

In the absence of clear information, 15% of projected average daily design flow can be used, or 25% of (Peak Balancing + Fire Flow).

3.6.5.4. Dead Storage

If a storage structure is of a type where only the upper portion of the water provides a useful function, such as maintaining usable system pressure, the remaining lower portion is considered dead storage. Dead storage can be considered useful if pumps can withdraw the water from the lower portion of the storage structure during a fire or other emergency. Where dead storage is present there must be adequate measures taken to circulate the water through the tank to maintain quality and prevent freezing (i.e. baffles, loading/unloading techniques, and adequate mixing provisions). Unusable dead storage should be avoided wherever possible.

3.6.5.5. Turnover and Water Quality

Deterioration in water quality is frequently associated with the age of the water. Loss of disinfection residual, formation of DBPs, and bacterial re-growth can all result from aging of water. As a result, an implicit objective in both design and operation of distribution system storage facilities is the minimization of detention time and the avoidance of volumes of water that remain in the storage facility for long periods. The allowable detention time should depend on the quality of the water, its reactivity, the type of disinfectant used and the travel time before and after the water's entry into the storage facility. A maximum 72-hour turnover is a reasonable guideline. If it is not possible to have sufficient turnover of water in the storage facility, supplemental disinfection may be required.

In cases where taste and odour problems exist and/or where excessive levels of DBPs are generated, bleeding at the ends of the system is a recommended measure provided that the chlorine levels are neutralized prior to discharge to receiving waters.

3.6.5.6. Plant Storage

The designer should recognize the need to calculate, in addition to distribution storage requirements, the requirement for the operation of the water treatment facility (i.e. backwash and domestic use).

3.6.5.6.1. Clearwell Storage

Clearwell storage should be sized, in conjunction with distribution system storage, to avoid frequent on/off cycling of the treated water pumps. A minimum of two compartments along with adequate measures for circulation should be provided. Clearwells that can be depleted should not be used to achieve the required chlorine contact times. A separate contact tank should be provided to meet the disinfection requirements.

3.6.6. Location of Distribution Storage

The location of distribution storage is closely associated with the system hydraulics and water demands in various parts of the system. Location of the storage facilities at natural high points within the area being serviced by the water system allows for gravitational advantage and potential considerable cost savings. The site selection process is often also affected by the availability of appropriate land and public acceptance of the structure.

3.6.6.1. Elevated Storage

Elevated Storage includes elevated tanks and the upper portion of water stored within standpipes. Elevated storage facilities that have existed for several years rarely bother the public, however, property owners will often object to a new one being built near their homes. Designs can be very pleasing and landscaping and colours can be used to minimize or even enhance the visual effect. This may not however be enough to overcome the objections of the local community and it may be necessary to build water elevated storage facilities at non-ideal locations from both topographic and hydraulic perspectives. Industrial zones may provide some opportunities, otherwise alternative facilities using above ground and in-ground water storage and pumps may be required.

3.6.6.2. Above Ground and In-Ground Storage Reservoirs

Low level above ground and in-ground storage reservoirs are generally used where a large quantity of water must be stored. A relatively large parcel of land is required to accommodate both the reservoir and the accompanying pump station.

The following are considered minimum requirements:

1. The bottom of above ground reservoirs and standpipes should be placed at the normal ground surface and should be above maximum flood level based on a 100-year flood;

2. When the bottom of the storage reservoir must be below normal ground surface, the in-ground reservoir should be placed above the groundwater table. Typically at least 50% of the water depth should be above grade. Sewers, drains, standing water, and similar sources of possible contamination must be kept at least 15 m from the reservoir; and

3. The top of an in-ground reservoir should not be less than 600 mm above normal ground surface. Clearwells constructed under filters may be exempted from this requirement when the total design gives the same protections.

3.6.7. Facility Requirements

3.6.7.1. Inlet/Outlet and Baffle Wall

A detailed design of the inlet, and outlet and, if required, baffle walls, mixing, etc., is required to ensure maximum turnover of water in a storage tank.

3.6.7.2. Level Control

Adequate controls should be provided to maintain levels in distribution system storage structures. Level indicating devices should be provided at a central location. Key issues are:

1. Pumps should be controlled from tank levels with the signal transmitted by telemetry equipment when any appreciable head loss occurs in the distribution system between the source and the storage structures;

2. Altitude valves or equipment controls are required to control pump on-off cycles or gravity flow to and from the tank to maintain the system pressures and avoid overflows;

3. Overflow and low-level warnings or alarms should be located at places in the community where they will be under responsible surveillance 24 hours a day; and

4. Changes in water level in a storage tank during daily domestic water demands should be limited to a maximum 9 m to stabilize pressure fluctuations within the distribution system.

3.6.7.3. Overflow

All above ground water storage structures should be provided with an overflow, which is brought to an elevation between 300 mm and 600 mm above the ground surface, and discharges over a drainage inlet structure or a splash plate. An overflow shall not be connected directly to a sewer or storm drain. All overflow pipes should be located so that any discharge is visible.

When an internal overflow pipe is used on elevation tanks, it should be located in the access tube. For vertical drops on other types of storage facilities, the overflow pipe should be located on the outside of the structure.

The overflow of a ground-level structure should open downward and be screened with 24-mesh non-corrodible screen installed within the pipe at a location least susceptible to damage by vandalism. Overflows should be located at sufficient elevation to prevent the entrance of surface water. A backflow preventer should be installed on all overflows, on in-ground or low-elevation reservoirs.

The overflow pipe should be of sufficient diameter to permit the wasting of water in excess of the filling rate.

Consideration should be given to downgrade receiving areas of overflow water. Adequate surface detention should be provided to prevent soil erosion and to provide safe dissipation of chlorine.

The discharge must not be directed to natural water bodies. Discharge in residential areas should be contained to appropriate and controlled storm water channels.

3.6.7.4. Drainage of Storage Structures

Water storage structures, which provide pressure directly to the distribution system, should be designed so they can be isolated from the distribution system and drained for cleaning or maintenance without necessitating loss of pressure in the distribution system. The drain should discharge to the ground surface with no direct connection to a sewer or municipal storm drain, and should be located at least 300 mm above ground surface.

Water that is drained from storage structures should be dechlorinated prior to discharge to the environment.

3.6.7.5. Roof Drainage

The roof of the storage structure should be well drained. Downspout pipes should not enter or pass through the reservoir. Parapets, or similar construction, which would tend to hold water and snow on the roof, should be avoided.

3.6.7.6. Roof and Sidewall

The roof and sidewall of all structures must be watertight with no opening except properly constructed vents, manholes, overflows, risers, drains, pump mountings, control ports, or piping for inflow and outflow.

1. Any pipes running through the roof or sidewall of a treated water storage structure must be welded, or properly gasketed in metal tanks. In concrete tanks, these pipes should be connected to standard wall castings, which were poured in place during the forming of the concrete. These wall castings should have seepage rings imbedded in the concrete;

2. Openings in a storage roof or top, designed to accommodate control apparatus or pump columns, should be curbed and sleeved with proper additional shielding to prevent the access of surface or floor drainage water into the structure;

3. Valves and controls should be located outside the storage structure so that the valve stems and similar projections will not pass through the roof or top of the reservoir; and

4. The roof of concrete reservoirs with earthen cover should be sloped to facilitate drainage. Consideration should be given to the installation of an impermeable membrane roof covering.

3.6.7.7. Vents

Finished water storage structures should be vented. Overflows should not be considered as vents. Open construction between the sidewall and roof is not permissible. The requirements for vents are as follows:

1. They should prevent the entrance of surface water and rainwater;

2. They should exclude birds and animals;

3. They should exclude insects and dust, as much as this function can be made compatible with effective venting. For elevated tanks and standpipes, 24-mesh non-corrodible screen may be used; and

4. They should, on ground-level structures, terminate in an inverted U construction with the opening 600 mm to 900 mm above the roof or sod and covered with 24-mesh non-corrodible screen installed within the pipe at a location least susceptible to vandalism.

3.6.7.8. Frost Protection

All finished water storage structures and their appurtenances, especially the riser pipes, overflows, and vents, should be designed to prevent freezing which may interfere with proper functioning.

3.6.7.9. Internal Catwalk

Every catwalk over finished water in a storage structure should have a solid floor with raised edges so designed that shoe scrapings and dirt will not fall into the water.

3.6.7.10. Silt Stop

The discharge pipes from all reservoirs should be located in a manner that will prevent the flow of sediment into the distribution system. Removable silt stops should be provided.

3.6.7.11. Grading

The area surrounding a ground-level structure should be graded in a manner that will prevent surface water from standing within 15 m of the structure.

3.6.7.12. Corrosion Prevention/Reduction

Proper protection should be given to metal surfaces by paints or other protective coatings, by cathodic protective devices, or by both.

1. Paint systems should meet AWWA Standard D102 and NSF Standard 61, and be acceptable to the DOEC. Interior paint must be properly applied and cured. After curing, the coating should not transfer any substance to the water that will be toxic or cause taste or odours. Prior to placing in service, an analysis for volatile organic compounds is advisable to establish that the coating is properly cured. Consideration should be given to 100% solid coatings.

2. Wax coatings for the tank interior should not be used on new tanks. Recoating with a wax system is discouraged, however, the old wax coating must be completely removed to use another tank coating.

3. Cathodic protection should be designed and installed by qualified technical personnel and a maintenance contract should be provided.

3.6.7.13. Disinfection

1. Finished water storage structures should be disinfected in accordance with current AWWA Standard C652. Two or more successive sets of samples taken at 24-hour intervals, should indicate microbiologically satisfactory water before the facility is placed into operation.

2. Disposal of heavily chlorinated water from the tank disinfection process should be in accordance with the requirements of the DOEC.

3. A disinfection procedure (AWWA Standard C652 chlorination method 3, section 4.3) which allows use of the chlorinated water held in the storage tank for disinfection purposes is recommended only where conditions warrant (i.e. where water supply is not abundant, or where large reservoirs would require excessive volumes of water and chlorine. The use of the heavily chlorinated water (depending on the system) may introduce various chlorinated organic compounds into the distribution system.

3.6.7.14. Provisions for Sampling

Appropriate sampling points should be provided to facilitate collection of water samples for both bacteriologic and chemical analyses.

3.6.7.15. Adjacent Compartments

Finished water must not be stored or conveyed in a compartment adjacent to unsafe water when a single separates the two compartments.

3.6.7.16. Basins and Wet-wells

Receiving basins and pump wet-wells for finished water should be designed as finished water storage requirements.

3.6.7.17. Standby Power

The necessity for standby power for a storage facility with pump discharge is dependent on whether the normal power is considered secure. In addition, the volume of elevated storage should be assessed when considering the requirements for standby power.

3.6.8. Water Treatment Plant Storage

3.6.8.1. Backwash Tanks

Backwash tanks should be sized, in conjunction with available pump units and finished water storage to provide the required filter backwash water. Consideration should be given to the backwashing of several filters in succession.

3.6.8.2. Clearwell

Clearwell storage should be sized, in conjunction with distribution system storage, to relieve the filters from having to follow fluctuations in water use.

1. When finished water storage is used to provide contact time for chlorine, special attention must be given to size and baffling;

2. If used to provide chlorine contact time, sizing of the clearwell should include extra volume to accommodate depletion of storage during the night time for intermittently operated filtration plants with automatic high service pumping from the clearwell during non-treatment hours;

3. A minimum of two clearwell compartments should be provided;

4. The overflow pipe should be of sufficient diameter to permit the wasting of water in excess of the filling rate;

5. Consideration should be given to downgrade receiving areas of overflow water. Adequate surface detention should be provided to prevent soil erosion and to provide safe dissipation of chlorine; and

6. The discharge must not be directed to natural water bodies. Discharge in residential areas should be contained to appropriate and controlled storm water channels.

3.6.8.3. Adjacent Compartments

Finished water must not be stored or conveyed in a compartment adjacent to unsafe water when a single wall separates the two compartments.

3.6.8.4. Wet-wells

Receiving pump wet-wells for finished water should be designed as finished water storage structures.

3.6.9. Hydropneumatic Tanks

The use of Hydropneumatic (pressure) tanks, as storage facilities is preferred for small water supply systems. When serving more than 150 living units, however, ground or elevated storage is recommended in accordance with sizing requirements as outlined in Section 3.6.4.

Pressure tank storage is not to be considered for fire protection purposes.

Pressure tanks should meet ASME code requirements or an equivalent requirement of provincial and local laws and regulations for the construction and installation of unfired pressure vessels.

3.6.9.1. Location

The tank should be located above normal ground surface and be completely housed.

3.6.9.2. Sizing

1. The capacity of the wells and pumps in a hydropneumatic system should be at least ten times the average daily consumption rate. The gross volume of the hydropneumatic tank in litres, should be at least ten times the capacity of the largest pump, rated in litres per minute. For example, a 750 L/min pump should have a 7500 L pressure tank; and

2. Sizing of hydropneumatic storage tanks should consider the need for chlorine detention time, if applicable.

3.6.9.3. Piping

The tank should have bypass piping to permit operation of the system while it is being repaired or painted.

3.6.9.4. Appurtenances

Each tank should have a drain, and control equipment consisting of pressure gauge, water sight glass, automatic or manual air blow-off, means for adding air, and pressure operated start-stop controls for the pumps. In large tanks, where practical, an access manhole should be 600 mm in diameter.

3.6.10. Security/Safety

3.6.10.1. Access

Only trained and experienced workers should be allowed to work in water storage facilities.

Finished water storage structures should be designed with reasonably convenient access to the interior for cleaning and maintenance. For in-ground tanks at least two manholes should be provided above the waterline at each water compartment where space permits.

Access manholes in above ground structures should be framed at least 100 mm above the surface of the roof at the opening. For below ground structures access, manholes should be elevated a minimum 600 mm above the top of covering sod;

1. Each of the manhole should be fitted with a solid watertight cover which overlaps the framed opening and extends down around the frame at least 50 mm;

2. Hinged at one side; and

3. Have a locking device.

3.6.10.2. Safety

The safety of employees must be considered in the design of the storage structure. As a minimum, such matters should conform to pertinent laws and regulations of the areas where the reservoir is constructed.

1. Ladders, ladder guards, offset balconies, balcony railings, and safety located entrance hatches should be provided where applicable;

2. Elevated tanks with riser pipes over 200 mm in diameter should have protective bars over the riser openings inside the tanks; and

3. Railings or handholds should be provided on elevated tanks where persons must transfer from the access tube to the water compartment.

3.6.10.3. Protection

All finished water storage structures should have suitable watertight roofs that exclude birds, animals, insects, and excessive dust.

Fencing, locks and access manholes, and other necessary precautions should be provided to prevent trespassing, vandalism, and sabotage as per AWWA standards.

3.7. Watermains

Water distribution systems are made up of pipe, valves, and pumps through which treated water is moved from the treatment plant to domestic, industrial, commercial, and other customers. The distribution system also includes facilities to store water, meters to measure water use, fire hydrants and other appurtenances. The major requirements of a distribution system is to supply each customer with sufficient volume of treated water at an adequate service pressure.

3.7.1. Definitions

Transmission Main – A transmission main is the pipeline used for water transmission, that is, movement of water from the source to the treatment plant and from the plant to the distribution system.

Transmission mains typically do not have service connections

Primary Distribution Main – A primary distribution main is a principal supply pipeline within a distribution system. A primary distribution main can also transport water to adjacent distribution networks.

Distribution Main – A distribution main is the local supply pipeline in the distribution system.

Service Line (Lateral) – A service line is the pipe (and all appurtenances) that runs between the utility's watermain and the customer's place of use, including fire lines.

Service Connection – A service connection is the portion of the service line from the utility's watermain to the curb stop at or adjacent to the street line or the customer's property line.

Average Day Demand – The largest daily rate of flow of water in a year that must be supplied by the water system to meet customer's demands.

Maximum Day Demand – The largest daily rate of flow of water in a year that must be supplied by a water system, to meet customer demand.

Peak Hour Demand – The largest hourly rate of flow of water in a year that must be supplied by a water system to meet customer demand.

Minimum Hour Demand – Can also be referred to as the night demand. It is the lowest hourly rate of flow of water in a year that must be supplied to meet customer demand.

Instantaneous Peak Demand – A short duration high water flowrate that can occur in a water supply system.

3.7.2. Materials

There are a variety of materials in use within water transmission and distribution systems. Typical water pipe material used include:

1. Ductile Iron;
2. Polyvinyl Chloride (PVC);
3. High Density Polyethylene (HDPE); and
4. Concrete Pressure Pipe.

3.7.2.1. Standards, Materials Selection

Pipe, fittings, valves and fire hydrants should conform to the latest standards issued by the CSA, AWWA, NSF, or NFPA, and be acceptable to the DOEC.

The proper selection of water pipe material should take into consideration the following:

1. Working pressure rating;
2. Surge pressure rating;
3. Internal and external corrosion resistance;

4. Negative pressure capability;
5. Ease of installation;
6. Availability;
7. Pipe rigidity with regards to trench conditions; and
8. Ease of repair.

3.7.2.2. Used Materials

Watermains, which have been used previously for conveying potable water, may be reused provided they meet the above standards and have been restored practically to their original condition.

3.7.2.3. Joints

Packing and jointing materials used in the joints of pipe should meet the standards of the CSA/AWWA and the DOEC. Pipe having mechanical joints or plain ends in combination with couplings having slip-on joints with rubber gaskets is preferred. Lead-tip gaskets should not be used. Repairs to lead-joint pipe should be made using alternative methods. Flanged joints should only be used I conjunction with fitting such as valves within a properly constructed chamber.

3.7.2.4. Corrosion Prevention/Reduction

Special attention should be given to selecting pipe materials that will protect against both internal and external pipe corrosion. All products should comply with CSA/ANSI standards.

If soils are found to be aggressive, and the choice of materials is limited and subject to corrosion, action should be taken to protect the watermain and fittings by encasement (wraps, coatings, etc.) and/or provision of cathodic protection. For small copper pipes, sacrificial anodes are recommended.

The design and installation of watermain encasements and cathodic protection should be as per the manufacturer's recommendations.

For water distribution systems with a known corrosion problem or that have a calculated Langelier Index of -2 or below, measures must be implemented to adjust pH levels and eliminate internal corrosion as a result of aggressive water. This is usually accomplished by dosing sufficient amounts of lime in the form of soda ash to buffer the water alkalinity.

3.7.3. Design Criteria – Transmission and Distribution Systems

3.7.3.1. Transmission and Distribution Pipelines

Transmission mains in water supply systems are typically large diameter, carry large flows under high pressure and are long in length, therefore the design activities should address the following:

1. Sizing for ultimate future design flows;
2. Sizing and layout to ensure adequate supply and turnover of water storage facilities;
3. Elimination of customer service take-offs;
4. Minimization of branch take-offs to help maintain flow and pressure control;

5. Air relief at high points and drain lines at low points;
6. Isolation valving to reduce the length of pipe required to be drained in a repair or maintenance shut-down;
7. Potential transient pressures; and
8. Master metering.

Primary distribution mains typically receive flow from transmission mains or pressure control facilities (booster pumps or pressure reducing valve) and supplies water to one or several local distribution systems as well as services to customers. The primary distribution main provides a significant carrying capacity or flow capability to a large area. Key design activities should address:

1. Implementing a minimum “dual” feed system of primary distribution mains to supply large distribution systems;
2. Looping and isolation valving to maintain services with alternate routing in the event of repair or maintenance shutdown.
3. Area metering;
4. Air relief at significant high points;
5. Sizing for future extensions; and
6. Elimination of dead-ends.

Distribution mains typically provide the water service to customers through a network of pipelines feed by the primary distribution mains. Key design activities should address:

1. Looping and isolation valving to maintain service with alternate routing in the event of repair or maintenance shutdown;
2. Adequate valving to provide an efficient flushing program;
3. Elimination of dead-ends; and
4. Pressure Surge Relief (requirements can be addressed by storage in the distribution system or other acceptable methods).

3.7.3.2. Water Demands

Where values for maximum day demand, peak hour demand, and minimum hour (night) demand are not known they can be derived using peaking factors (i.e. applying numerical ratios of the average day demands).

Wherever possible, peaking factors based on actual usage records for a given water supply system should be used in the hydraulic analysis of a water transmission and distribution system. If however such records do not exist or are unreliable, Table 3.7 can be used as a guide.

The peaking factors contained in Table 3.7 are suitable for use in the hydraulic analysis of a municipal system with a variety of uses (residential, public, commercial, industrial). For small water systems where water usage is strictly residential and there are no water usage records, then the Harmon Formula in conjunction with the theoretical water usage of 340 L/cap/day can be used. Water demands and peaking factors for systems containing appreciably large areas of commercial or industrial lands will require an evaluation of water demands based on individual facility users.

Table 3.7
Peaking Factors for Municipal Water Supply Systems

Equivalent Population	Minimum Hour Factor	Maximum Day Factor	Peak Hour Factor
500 to 1000	0.40	2.75	4.13
1001 to 2000	0.45	2.50	3.75
2001 to 3000	0.45	2.25	3.38
3001 to 10 000	0.50	2.00	3.00
10 001 to 25 000	0.60	1.90	2.85
25 001 to 50 000	0.65	1.80	2.70
50 001 to 75 000	0.65	1.75	2.62
75 001 to 150 000	0.70	1.65	2.48
Greater than 150 000	0.80	1.50	2.25

3.7.3.3. Pressure

All transmission mains, primary distribution mains, distribution mains and service mains, including those not designed to provide fire protection, should be sized based on results of a hydraulic analysis of flow demands and pressure requirements.

Transmission and distribution mains should be designed to withstand the maximum working pressure plus pressure surge allowance. Mains should be tested to 1.5 times the working pressure, within a minimum of 520 kPa (75 psi) and maximum of 1200 kPa (175 psi).

The transmission and distribution system should be designed to maintain a minimum pressure of 275 kPa (40 psi) at ground level at all points in the distribution system under normal flow conditions.

Fire flow residual pressure should be maintained at 150 kPa (22 psi) at the flow hydrant, and should be a minimum 140 kPa (20 psi) within the system, for the design duration of the fire flow event.

The normal working pressure in the distribution system should be 410 kPa to 550 kPa (60 psi to 80 psi). The maximum design pressure during minimum demand periods should not exceed 650 kPa (95 psi).

3.7.3.4. Diameter

The minimum nominal diameter of pipe should be as follows:

1. 200 mm for primary distribution mains (300 mm is recommended);
2. 150 mm for distribution mains; and
3. 150 mm for service mains providing fire protection.

3.7.3.5. Small Mains for Domestic Services

1. Small mains for domestic services are acceptable for use in systems not required to carry fire flows;

2. The minimum size of a watermain in a distribution system where fire protection is not to be provided should be a minimum of 75 mm in diameter;

3. Watermains beyond the last hydrant on cul-de-sacs or dead end roads can have pipe sizes from 50 mm down to 25 mm diameter. For water service connections the minimum pipe size required is 20 mm inside diameter;

4. Any departure from the minimum requirements shall be justified by hydraulic analysis and future water use, and can be considered only in special circumstances; and

5. Watermains not designed to carry fire flows are not allowed to be connected to a fire pumper.

3.7.3.6. Velocity

The maximum design velocity for flow under maximum day conditions for transmission mains, primary distribution mains, distribution mains and service mains should be 1.5 m/s. The maximum fire flow velocity should be 3.0 m/s.

Flushing devices should be sized to provide a flow that provides a minimum cleansing velocity of 0.75 m/s in the watermain being flushed.

3.7.3.7. Dead Ends/Looping Requirements

Water distribution systems should be designed to exclude any dead-ended primary distribution mains, and distribution mains unless unavoidable. Appropriate tie-ins (loops) should be made wherever practical.

Where dead-end mains occur, they should be provided with a fire hydrant if flow and pressure are sufficient, or with an approved flushing hydrant or blow-off for flushing purposes. Flushing device shall not be directly connected to any sewer.

3.7.3.8. Fire Protection

All transmission mains, primary distribution mains and distribution mains, including those designed to provide fire protection, should be sized based on a hydraulic analysis to be carried out to determine flow demands and pressure requirements. The minimum size of watermain for providing fire protection and serving fire hydrants should be 150 mm diameter.

3.7.3.9. Fire Pumps

NFPA 20 covers the selection of stationary pumps and installation of pumps supplying water for private fire protection. Items include:
1. Water supplies;
2. Suction;

3. Discharge;
4. Auxiliary equipment;
5. Power supplies;
6. Electric drive and control;
7. Internal combustion engine drive and control;
8. Steam turbine drive and control; and
9. Acceptance tests and operation.

Stored water may be required to meet the demand for fire protection for a given duration. A reliable and "safe" method of replenishment would be required (See Section 3.6).

3.7.3.10. Drain/Flushing Devices

Drain/flushing devices should be placed at significant low points in the transmission system. The drain/flushing devices are required to accommodate flushing during construction, and after a watermain break to drain the pipe for repair.

Where flushing devices are to be installed, they are to be designed in accordance with the requirements of AWWA C651 and due care with respect to: dechlorinating; exit velocity of water during flushing (potential erosion/scour); minimum separation distance from nearest watercourse; storage etc.

Flushing device shall not be directly connected to any sewer.

3.7.3.11. Valves

Valves to be used in water distribution systems shall be manufactured in accordance with recognized standards, such as those prepared by AWWA and as covered in the *Municipal water, Sewer and Road Specifications*.

3.7.3.12. Valve Location

Sufficient valves shall be provided on watermains so that inconvenience and sanitary hazards will be minimized during repairs. Valves should be located at not more than 150 m intervals in commercial districts and at not more than one block or at not more than 240 m intervals in other districts. Where systems serve widely scattered customers and where future development is not expected, the valve spacing should not exceed 1.6 km. In grid patterns, intersecting watermains should be equipped with shut-off valves, as follows, to minimize disruption during repairs:

1. T intersection – at least 2; and

2. Cross intersection – at least 3.

3.7.3.13. Air Relief and Vacuum Valves

Air relief and vacuum valves should be installed, in a chamber, at significant high points in the transmission system and at other such locations as required for efficient operation of the water system.

Automatic air relief valves should not be used in situations where flooding of the manhole or chamber may occur.

The open end of an air relief pipe from automatic valves larger than 50 mm diameter should be extended at least 2.5 m above grade and provided with a screened and downward-facing elbow. The pipe from a manually operated valve should be extended to the top of the air relief chamber.

3.7.3.14. Flow Monitoring

Flow monitoring devices and flow meters should be positioned at key locations along the transmission and primary distribution mains.

3.7.3.15. Crossing Obstacles

Due to geography, parallel services, etc., there will be a variety of physical obstacles, which will result in the watermain crossing obstacles. Considerations include, but are not limited to the following: road crossings, sewers, surface water crossings, and horizontal drillings.

3.7.3.15.1. Road Crossings

It is recommended for all new watermains crossing existing roads and all new roads crossing existing watermains that there is:

1. A minimum cover of 1.8 m from the top of the pipe;
2. Backfill method and material is approved;
3. Drainage is adequate; and
4. Ditches crossing watermains should provide minimum cover of 1.8 m or insulate for frost protection.

3.7.3.15.2. Sewers

See Section 3.7.6.

3.7.3.15.3. Surface Water Crossings

Surface water crossings, whether over or underwater, require special considerations. The DOEC should be consulted before final plans are prepared.

The pipe should be adequately supported and anchored, protected from damage and freezing, and accessible for repair or replacement.

A minimum ground cover of 600 mm should be provided over the pipe. When crossing watercourses, which are greater than 4.5 m in width, the following should be provided:

1. The pipe should be of special construction, having flexible, restrained or welded watertight joints;
2. Valves shall be provided at both ends of water crossings so that section can be isolated for testing or repair; the valves should be easily accessible, not subject to flooding and should be within a properly constructed chamber.

3.7.3.15.4. Horizontal Drillings

Other methods of installation of watermains crossing obstacles or in deep installations include horizontal drilling/boring and installing pipe sections in protective sleeves.

3.7.3.16. Bedding

Bedding material and methodology should be as outlined in Section 02223 of the *Municipal Water, Sewer and Road* Specifications, and should be no less than as recommended by the pipe manufacture.

Do not lay pipe and fittings when the trench bottom is frozen, underwater or when trench conditions or weather are unsuitable.

3.7.3.17. Cover

All watermains shall have a minimum depth of cover not less than the depth of frost penetration, or a minimum of 1.8 m, whichever is greater. If this is not possible, then insulation around the pipe is required. In addition there is a requirement to have sufficient cover over watermains to minimize mechanical loading (See Section 3.7.3.15.1.). It is also recommended that maximum allowable depth be specified.

3.7.3.18. Warning/marker and Detection Tape

Warning/marker and detection tape as specified in the Department of Municipal and Provincial Affairs Water, Sewer and Roads Master Specification Section 02223.2.1 and detailed drawings numbered 0290 and 0300, shall be installed continuously with a minimum 1.0 m overlap at joints above water, sewer, and forcemains. Warning/marker tape shall be heavy gauge polyethylene, 150 mm wide and indicate the service line below. Detectable tape shall 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 300 mm below ground surface and be detectable using conventional pipe location apparatus.

3.7.3.19. Thrust Restraint

All tees, bends, plugs and hydrants should be provided with reaction blocking, tie rods or restrained joints designed to prevent movement.

In situations where a watermain installation is above deep fills or parallel to a deep sewer, consideration should be given to using restrained joints.

3.7.3.20. Pressure and Leakage Testing

All types of installed pipe should be pressure tested and leakage tested in accordance with the latest edition of AWWA Standard C600, and as outlined in Section 02713 of the *Municipal Water, Sewer and Road Specifications*.

3.7.3.21. Disinfection

All new, cleaned or repaired watermains should be disinfected in accordance with the latest AWWA Standard C651. The specifications should include detailed procedures for the adequate flushing, disinfection, and microbiological testing of all watermains. In an emergency or unusual situation, the disinfection procedure should be discussed with the DOEC.

3.7.3.22. Commissioning

Following successful testing and disinfection of watermains, the new system should be commissioned with due consideration of resulting pressure and flow changes and other parameters that may be experienced within the water supply system.

3.7.4. Hydrants

Hydrants shall conform to the latest AWWA Standard C502, and shall be ULC and FM approved.

All fire hydrants and flush hydrants should be of "self-draining" Dry Barrel type. In areas having high water tables, appropriate measures should be taken to ensure drainage of the hydrant barrel (pumping or other suitable means).

Watermains which are not designed to carry fire-flows should not have fire hydrants connected to them.

3.7.4.1. Location and Spacing

Hydrants should be provided at each street intersection and at intermediate points between intersections as recommended by the Insurance Advisory Organization (IAO) and the Fire Commissioners Office. In the absence of clear guidance hydrant spacing may range from 100 m to 175 m depending on the area being served and in accordance with IAO and Fire Commissioners Office requirements.

3.7.4.2. Valves and Nozzles

Valves and nozzles shall be as outlined in Section 02713 of the *Municipal Water, Sewer and Roads* Specifications.

Specific requirements should be coordinated with the local fire authority.

3.7.4.3. Hydrant Leads

The hydrant lead should be minimum of 150 mm in diameter. Shut-off valves should be installed in all hydrant leads.

3.7.4.4. Drainage

Attention must be given to drainage of sub-surface hydrant chambers, and only where unavoidable, should pumping chambers dry be specified. Where this is required the hydrants must be clearly marked as non-draining.

Hydrants may also require to be pumped dry when hydrant drains are plugged during freezing weather.

Hydrant drains consisting of a gravel pocket or dry well should be provided unless the natural soils will provide adequate drainage.

Hydrant drains should not be connected to or located within 3 m of sanitary sewers or storm drains.

2.7.4.5. Frost Protection

No type of antifreeze product will be permitted for use as a frost protection for dry barrel fire hydrants situated in high water table areas. An alternative is to plug drain outlets with approved plugs and to drain the hydrant seasonally and after use by pumping out the hydrant.

3.7.5. Valve and Metering Chambers

3.7.5.1. Chamber Construction

Chambers for air relief and vacuum valves, flow monitoring/measuring devices and pressure reducing valves should be:

1. Constructed to provide a watertight structure with easy and safe access;
2. Designed to include watertight gaskets where a pipe passes through a chamber wall; flexible rubber “A-Luk” type for cast-in-place concrete or mechanical expansion insert type for pre-cast concrete;
3. Insulated to ensure adequate frost protection; and
4. Include gravity or pump drainage.

3.7.5.2. Air Relief and Vacuum Valve Chambers

Air relief and vacuum valves should be installed, in a chamber, at significant high points in the distribution system and at other such locations as required for efficient operation of the water system.

Automatic air relief valves should not be used in situations where flooding of the manhole or chamber may occur.

3.7.5.3. Flow Measurement and Meter Chamber

Chambers containing flow monitoring/measurement devices should be located at off-road locations where feasible.

3.7.5.4. Pressure Reducing Valve Chambers

Pressure reducing valve chambers should be designed and constructed to provide:

1. By-pass capability;
2. Isolation valves on the upstream and downstream piping for the pressure reducing valve; and
3. Upstream and downstream pressure gauges.

3.7.5.5. Chamber Drainage

Chambers should be drained, if possible, to the surface of the ground where they are not subject to flooding by surface water, or to underground absorption pits. Drains should be equipped with a backflow prevention device and screening to prevent the entry of insects, birds, and rodents.

In areas where high ground water levels are evident, above water table chambers should be considered.

3.7.6. Separation Distances to Sanitary and Storm Sewers

The following factors should be considered in providing adequate separation:

1. Materials and type of joints for water and sewer pipes;
2. Soil Conditions;
3. Service and branch connections into the watermain and sewer line;
4. Compensating variations in the horizontal and vertical separations;
5. Space for repair and alterations of water and sewer pipes; and
6. Offsetting of pipes around manholes.

3.7.6.1. Parallel Installation

Watermains should be laid at least 3 m horizontally from any existing or proposed sewer. The distance should be measured edge to edge. In cases where it is not practical to maintain a 3 m separation, the DOEC may allow deviation on a case-by-case basis, if supported by data from the design engineer. Such deviation may allow installation of the watermain closer to a sewer, provided that:

1. The watermain is laid in a separate trench, or on an undisturbed earth shelf located on one side of the sewer; and
2. At such an elevation that the bottom of the watermain is at least 450 mm above the top of the sewer, and 300 mm horizontal measured edge-to-edge, or as required by the DOEC.

If separate trenches are used then the soil between the trenches must be undisturbed.

3.7.6.2. Crossings

Watermains crossing sewers should be laid to provide a minimum vertical distance of 450 mm between the outside of the watermain and the outside of the sewer. This should be the case where the watermain is either above or below the sewer with preference to the watermain located above the sewer. At crossings, above or below, one full length of water pipe should be located so both joints will be as far from the sewer as possible. Special structural support for the water and/or sewer pipes may be required.

3.7.6.3. Forcemains

There should be at least 3 m horizontal separation between watermains and sanitary sewer forcemains. When crossing, the watermain should be above the forcemain with a vertical separation of a minimum 450 mm at the crossing.

Where it is anticipated that watermains and forcemains will conflict at the crossings, then the forcemain shall be lowered in order to achieve the minimum 450 mm separation.

The DOEC should be contacted in instances where existing infrastructure does not allow for the watermain to be placed above the forcemain at the required separation.

3.7.6.4. Manholes

Water pipe should not pass through or come on contact with any part of the sewer manhole.

3.7.6.5. Other Sources of Contamination

Design engineers should exercise caution when locating watermains at or near certain sites such as sewage treatment plants or industrial complexes. On site waste disposal facility including adsorption fields must be located and avoided. The engineer should establish specific design requirements for locating watermains near any source of contamination and coordinate planned activities with the DOEC.

3.7.6.6. Water Only Servicing

Where municipal sewers are not provided, watermains must not pass within 15 m of any part of an in-ground sewage disposal system. Water service lines must not pass within 7.5 m of a sewage disposal system. In general, the following conditions should be met in regards to water service lines:

1. There is no joint in the service line between the dwelling and the connection to the curb stop;

2. The groundwater level should not be above the service line; and

3. The service line should be placed upslope of the sewage disposal field.

If these conditions are not met, consideration should be given to increasing the distance between the service line and the sewage disposal system, providing extra protection against contamination.

3.7.6.7. Exceptions

The DOEC must specifically approve any variance from the above requirements when it is impossible to obtain the specified separation distances. Where sewers are being installed and the above requirements cannot be met, the sewer materials should be waterworks grade 1000 kPa (150 psi) pressure rated pipe or equivalent and should be pressure tested to ensure water tightness.

3.7.7. Cross-connection Control

3.7.7.1. Cross-connection Control Programs

There should be no connection between the distribution system and any pipes, pumps, hydrants, or tanks whereby unsafe water or other contaminating materials may be discharged or drawn into the system. A Cross Connection Program should be in place for detecting and eliminating cross connections.

Where there is a requirement for water from the distribution system to be used as part of a process/procedure involving contaminants, measures must be taken to have discontinuous systems (i.e. break-tanks with anti siphon filter pipes or fail-safe backflow devices.

3.7.7.2. Interconnections

The approval of the DOEC should be obtained for interconnections between separate potable water supplies.

3.7.7.3. Backflow Prevention

Backflow prevention devices should be installed on consumer service connections where there is a high risk of contamination of the potable water supply system resulting from backflow or backpressure.

3.7.8. Water Services and Plumbing

3.7.8.1. Plumbing

Water services and plumbing should conform to relevant local and/or provincial plumbing codes, or to the applicable National Plumbing Code. Solders and flux containing more than 0.2% lead and pipe and pipe fitting containing more than 8% lead should not be used.

3.7.8.2. Consumer Connections (Laterals and Curb-Stops)

All consumer connections (laterals) should conform to the following:

1. Minimum cover 1.6 m;
2. Maximum cover 2.0 m;
3. 300 mm minimum horizontal and vertical separation distance from gravity sewer pipes;
4. Minimum 450 mm vertical separation when crossing above a sewer pipe;
5. Minimum separation distance of 3.0 m from outdoor fuel tank;
6. Minimum separation from sewage disposal field of 7.5 m;
7. Single-family residence connections should be minimum 20 mm copper or 25 mm HDPE pipe. Large sizes may be required depending on length of lateral and grade elevations;
8. Solder and flux containing more than 0.2% lead should not be used;
9. Maximum velocity of flow should not exceed 4.5 m/s;
10. There should be not joint between the curb-stop and the building, if possible;
11. A shut-off valve (curb-stop) should be fitted on the street side of the property boundary;
12. An approved metering device should be fitted, where applicable;
13. Backflow prevention devices, when required, should be installed after metering device;
14. Shut-off valve should be fitted before the metering device; and
15. Pressure reducing valves to be fitted as required before metering device.

3.7.8.3. Booster Pumps

Individual booster pumps should not be used for any individual service from the public water supply mains unless approved by the DOEC.

3.7.8.4. Service Meters

Each service consumer connection should be individually metered with an approved metering device, where applicable.

3.7.8.5. Water Loading Stations

Water loading stations prevent special problems since the fill line may be used for filling both potable water vessels and other tanks or contaminated vessel. To prevent contamination of both the public supply and potable water vessels being filled, the following principles should be met in the design of water loading stations:

1. A reduced pressure principle backflow prevention device should be installed on all watermains supplying water loading stations;
2. The piping arrangement should prevent contaminant being transferred from a hauling vessel to other subsequently using the station;
3. Hoses should not be contaminated by contact with the ground;
4. A loading station should be designed to provide access only to authorized personnel; and
5. Access to a loading station should be strictly controlled to minimize water safety and security concerns.

3.7.8.6. Sampling Stations

Dedicated sampling station may be required, within a water transmission and/or water distribution system, to collect water samples as part of the water quality monitoring program. The need for, and the proposed locations of sampling stations, should be discussed with the DOEC.

Section 4 – Disinfection of Drinking Water

4. Disinfection of Drinking Water

The goal of water disinfection is the inactivation of microorganisms, such as viruses, bacteria and protozoa, which can cause serious illnesses and death. Although disinfection can be accomplished to a significant extent by a number of physiochemical water treatment processes, such as coagulation, sedimentation, filtration, lime-soda softening and adsorption, a specific chemical disinfection step is usually incorporated into surface water treatment process trains to prevent the transmission of waterborne diseases.

Continuous disinfection is mandatory for all public water supplies

4.1. Forms of Disinfection

Chlorine is the most commonly used chemical for disinfection. The forms most often used are chlorine gas and calcium or sodium hypochlorite. Alternative disinfectants include ozone, chlorine dioxide, chloramines, ultraviolet light, iodine, gamma irradiation, and others. The chemical or technology should be selected after due consideration of water flow rates, application and demand rates, pH of the water, cost of equipment, chemical availability, and maintenance problems.

4.2. Chlorination

4.2.1. Contact Time and Point of Application

1. Due consideration shall be given to the contact time of the disinfectant in water in relation to pH, ammonia, taste-producing substances, temperature, bacterial quality, DBP formation potential and other pertinent factors.

2. Chlorine application should be at a point, which will provide a contact time of at least 20 minutes at peak hourly flow (using the Harmon Formula or peaking table) with required free chlorine residual. All basins used for disinfection must be designed to minimize short-circuiting. Additional baffling can be added to new or existing basins to minimize short-circuiting and increase contact time.

3. The point of application shall be located in order to minimize the formation of DBPs without compromising the integrity of contact time.

4. Adding ammonia to the water shall be done only with the approval of the DOEC.

5. At plants treating surface water, provisions shall be made for applying the disinfectant to the raw water, filtered water and, water entering the distribution system. However, these practices should be optimized on a case-by-case basis in order to minimize DPB formation.

6. As a minimum, at plants treating groundwater, provisions shall be made for applying the disinfectant to the detention basin inlet and water entering the distribution system.

7. If primary disinfection is accomplished using ozone, chlorine dioxide, or some other chemical or process that does not provide a measurable residual disinfectant, then chlorine must be added as a secondary disinfectant to provide a residual disinfectant as mentioned in Section 4.2.3.

4.2.2. CT Factor and Log Inactivation

4.2.2.1. CT Factor

CT factor is one of the most important features for determining or predicting germicidal efficiency of any disinfectant. It is the best method to ensure that the water provided to customers is safe. Chemical disinfection does not remove microorganisms from water but inactivates them so they can no longer infect consumers. The CT factor is defined as the product of the residual disinfectant concentration, C in mg/L, and the contact time T, in minutes, that residual disinfectant is in contact with the water. CT tables have been developed that relate CT values to levels of inactivation under various operating conditions. Different tables exist for different disinfectants. As the CT value is increased, a greater percentage of microorganisms are inactivated by chemical disinfection. The CT, and therefore the level of inactivation, can be increased by, applying greater doses of the disinfectant or by increasing the time that the water is in contact with the disinfectant.

Various factors can affect CT values, such as pH, temperature, strength of disinfectants and types of organisms:

1. As pH increases, the CT value also needs to be increased. This can be explained by examining the effects of pH on free chlorine. As the pH increases, more of the weak disinfectant (OCl-) exists than the strong disinfectant (HOCl-), thus increasing the CT value;

2. With temperature, in general for all disinfectants, as temperature increases, effectiveness increases;

3. The strength of a disinfectant directly affects the CT. For a weak disinfectant, the CT will have to be higher than for a strong disinfectant; and

4. Different organisms have different resistances to disinfectants. If an organism has a strong resistance to a certain disinfectant, the CT will need to be higher than for an organism with a weaker resistance.

The first step in using CT Disinfection is to determine the required CT to demonstrate that enough disinfection is occurring. To determine CT, you need to know:

1. The minimum temperature of the water during disinfection. The minimum temperature of the water in the chlorine contact chamber must be monitored. Minimum temperature is used because chlorine's ability to disinfect becomes less with lower temperatures. By using the lowest temperature of the water when determining CT, we know that the disinfection that occurred was at least as good as the lowest temperature allowed.

2. The maximum pH of the water during disinfection. The maximum pH of the water in the chlorine chamber must be monitored. Chlorine's ability to disinfect becomes less as pH increases. By using the maximum pH when determining CT, we know that the disinfection that occurred was at least as good as the disinfection that occurs at the maximum pH.

3. The minimum chlorine residual in the water during disinfection. The minimum chlorine residual in the water must be monitored at the end of the disinfection chamber. We know that higher chlorine dosages disinfects better. The lowest chlorine residual is used because the water in the chlorine contact chamber has been exposed to at least that concentration of chlorine.

4. The Log reduction by disinfection must be known. The required Log reduction by disinfection is based on the raw water quality with allowances for the treatment at the water treatment plant.

With this information we use the CT tables to determine the CT required.

CT tables are used as follows:

1. Make sure you are using the correct table. The tables are specific to the target organism and the type of disinfectant. Most likely, you will use the *Giardia* inactivation table for free chlorine. Free chlorine is the most common disinfectant used. Free chlorine is more than ten times better at inactivating *Giardia* than chloramines.

2. The tables are temperature specific. You must use the table that corresponds to your measured minimum temperature.

3. The tables are divided into pH sections. Locate the section of the table that corresponds to your measured maximum pH.

4. Within the appropriate pH range, locate the disinfection Log inactivation for *Giardia*.

5. Read the CT value from the table where the chlorine residual row meets the required Log inactivation column.

6. CT is read directly from the table.

The CT tables are temperature specific for water temperatures at 5°C increments. If your temperature falls between two temperatures for which tables exist, for example 8°C, then you need to determine the CT by one of the following methods:

1. Determine the CT at both 5°C and 10°C from the corresponding tables and estimate the CT value for 8°C using the CT values for 5°C and 10°C; or

2. Use the CT table that exists for the next lower temperature. To determine CT for 8°C, use the CT value for 5°C. This will produce a value that is conservative (i.e. higher) and adds an extra measure of safety.

To determine the CT value for a pH that does not correspond to one of the given pH sections, the CT value can be estimated using the pH sections higher and lower than your measured pH or by using the CT value at the next higher pH section.

Tables 4.1, 4.2, 4.3 and 4.4, presented below, represent a few selected CT Tables for *Giardia* and viruses using free chlorine or chloramine.

Table 4.1
CT Values for 3.0 log (99.9%) Inactivation of *Giardia* Cysts by Free Chlorine
(Water Temperature of 15°C)

	pH						
Free Chlorine Residual (mg/L)	**≤ 6.0**	**≤ 6.5**	**≤ 7.0**	**≤ 7.5**	**≤ 8.0**	**≤ 8.5**	**≤ 9.0**
0.4	49	59	70	83	99	118	140
0.6	50	60	72	86	102	122	146
0.8	52	61	73	88	105	126	151
1.0	53	63	75	90	108	130	156
1.2	54	64	76	92	111	134	160
1.4	55	65	78	94	114	137	165
1.6	58	66	79	96	116	141	169
1.8	57	68	81	98	119	144	173
2.0	58	69	83	100	122	147	177
2.2	59	70	85	102	124	150	181
2.4	60	72	86	105	127	153	184
2.6	61	73	88	107	129	156	188
2.8	62	74	89	109	132	159	191
3.0	63	76	91	111	134	162	195

Table 4.2
CT Values for Inactivation of Viruses by Free Chlorine

	Log Inactivation					
Temperature	**2.0 Log**		**3.0 Log**		**4.0 Log**	
°C	**pH 6 - 9**	**pH 10**	**pH 6 - 9**	**pH 10**	**pH 6 - 9**	**pH 10**
0.5	6	45	9	66	12	90
5	4	30	6	44	8	60
10	3	22	4	33	6	45
15	2	15	3	22	4	30
20	1	11	2	16	3	22
25	1	7	1	11	2	15

Table 4.3
CT Values for Inactivation of *Giardia* Cysts by Chloramine
Within the pH Range 6 to 9

	Temperature (°C)					
Inactivation	**≤1**	**5**	**10**	**15**	**20**	**25**
0.5-log	635	365	310	250	185	125
1.0-log	1270	735	615	500	370	250
1.5-log	1900	1100	930	750	550	375
2.0-log	2535	1470	1230	1000	735	500
2.5-log	3170	1830	1540	1250	915	625
3.0-log	3800	2200	1850	1500	1100	750

Table 4.4
CT Values for Inactivation of Viruses by Chloramine

	Temperature (°C)					
Inactivation	**≤1**	**5**	**10**	**15**	**20**	**25**
2.0-log	1243	857	643	428	321	214
3.0-log	2063	1423	1067	712	534	356
4.0-log	2883	1988	1491	994	746	497

4.2.2.2. Log Inactivation or Removal

The level of inactivation is generally referred to in terms of log inactivation since inactivation is measured on a logarithmic scale (i.e. orders of magnitude reduction). Log inactivation is a measure of the percent of microorganisms that are inactivated during the disinfection process and is defined as:

$$Log\ Inactivation = Log\left(N_O / N_T\right)$$

where: N_O = initial (influent) concentration of viable microorganisms;
N_T = concentration of surviving microorganisms; and
Log = Logarithm to base 10.

Log inactivation is related to the percent inactivation, defined as:

$$Percent\ Inactivation = \left(1 - NT / NO\right) \times 100$$

Therefore, the relationship between log inactivation and percent inactivation is as follows:

$$Percent\ Inactivation = \left(1 - \frac{1}{10^{Log\ Inactivation}}\right) \times 100$$

or

$$Log\ Inactivation = Log\left(\frac{100}{100 - Percent\ Inactivation}\right)$$

Log Inactivation and Percent Inactivation are presented in Table 4.5.

Table 4.5
Log Inactivation and Percent Inactivation

Log Inactivation	Percent Inactivation
0.0	0.00
0.5	68.38
1.0	90.00
2.0	99.00
3.0	99.90
4.0	99.99
5.0	99.999
6.0	99.9999
7.0	99.99999

4.2.3. Residual Chlorine and Testing

1. All water entering a water distribution system, after a minimum 20 minutes contact time at peak hourly flow, shall contain a residual disinfectant concentration of free chlorine of at least 0.3 mg/L, or equivalent CT value. A detectable free chlorine residual must be maintained in all areas of the distribution system.

2. Higher residuals may be required depending on pH, temperature and other characteristics of the water.

3. Chlorine testing should include both free and total chlorine.

4. All systems, as a minimum, should use the DPD method that utilizes the digital readout with a self-contained light source. In this regard, the equipment shall enable measurement of chlorine residuals to the nearest 0.02 mg/L in the range of 0.02 to 8.0 mg/L, and shall be the filter photometer type with digital display readout, or equivalent accuracy, and shall be compatible with the equipment used by the DOEC and the Department of Government Services

4.2.4. Chlorine Gas - Facility Design

New facilities should be designed so that there are no pits, vaults or basements where chlorine equipment or piping would have to be installed in order to add chlorine to the water system.

4.2.4.1. Capacity

1. Chlorination equipment shall be sized to provide the specified free chlorine residual after 20 minutes of contact time at the maximum design flow rate. **It shall be designed to maintain proper chlorine residual under variable chlorine demand conditions, including peak flow.**

2. Each installation should have the capability to manifold several cylinders together to satisfy the peak demand and to avoid frequent replacement. No more than 2 tonne containers should ever be manifolded together.

4.2.4.2. Standby Equipment

1. Where chlorination is required for protection of the supply, standby equipment of sufficient capacity should be available to replace the largest unit.

2. Spare parts and tools shall be made available to replace parts subject to wear and breakage. Examples of spare parts are rubber fittings, hose clamps, gaskets and glassware.

3. If there is a large difference in feed rates between routine and emergency dosages, a gas metering tube should be provided for each dose range to ensure accurate control of the chlorine feed.

4.2.4.3. Automatic Switchover

Automatic switchover capability from an empty cylinder (or tonne container) to a full one should be provided in all installations.

4.2.4.4. Automatic Proportioning and Residual Analyzer

Where flow varies, an automatic flow proportional system should be installed. If chlorine demand varies then a residual analyzer with recorder should be installed. If both the flow and the chlorine demand vary, then a compound loop system should be installed.

4.2.4.5. Chlorination Room

1. All chlorination equipment should be installed in an above ground room that is separate from the general treatment plant so that the operator entering the facility or treatment plant would not be exposed to chlorine gas resulting from a gas leak.

2. Chlorine gas under pressure shall not be permitted outside the chlorine room. Chlorination systems under vacuum shall also be confined to the chlorine room with the exception of the chlorinator, which may or may not be located in the chlorine room. If the chlorinator is to be located outside the chlorine room it shall be located on the common wall, located as close as possible to the cylinders to minimize the length of the vacuum lines. The vacuum lines shall gently slope back towards the cylinders to facilitate the return of any condensate to the cylinders.

3. The chlorination equipment room should be designed with safety in mind and should generally incorporate the following features:

   The room shall be constructed in such a manner that all openings between the room and the remainder of the facility are sealed; and

   A reinforced glazed window should be installed in an exterior door or interior wall of the chlorination room to allow for visual inspection without having to enter the room. This window should not be able to be opened and must be airtight.

4. An outside door shall be provided to the room for easy escape in the case of gas leakage. Doors should be airtight, open outwards and be equipped with panic hardware. The door shall have mounted on its exterior, a sign "DANGER CHLORINE" warning of the presence of chlorine and a reminder to turn on the fan before entering.

5. Interior access to a non-chlorine area shall only be through a pressure-ventilated vestibule.

6. The distance from any point in the room to the exit door should not exceed 4.5 m.

7. The chlorinator should be placed at least 1 m from the outside wall.

4.2.4.6. Separate Storage Room

Where possible chlorine cylinders and containers should be stored in a room separate from the chlorination equipment room and the remainder of the facility and shall not be stored with combustible materials. The cylinder storage room shall be designed similar to the chlorine room.

The chlorine and/or cylinder storage rooms should be designed to facilitate the following:

1. Easy transfer of cylinders to and from the chlorination equipment room;

2. Easy handling of cylinders within the storage room in accordance with manufacturer's recommendations;

3. Empty and full cylinders should be stored separately in an upright position held by chains, straps, etc;

4. Cylinders (or tonne containers) should not be stored near flammable materials, heating or ventilation units, elevator shafts, or on uneven or subsurface floors;

5. Full chlorine cylinders should never be stored outside;

6. Tonne containers should be shielded from direct sunlight or from overheating above 60EC (140EF) from any source either while in storage or in use;

7. The number of openings in this room should be kept to a minimum to better control heating and ventilation.

4.2.4.7. Floor Drains

Floor drains <u>should not</u> be installed in either the chlorine equipment room or the cylinder storage room. If floor drains must be provided they are to discharge to a separate sump outside the building and shall not be connected to other internal or external drainage systems.

4.2.4.8. Construction Materials

1. The material of construction of these rooms should be non-corrodible and fireproof such as concrete, bricks, etc. to minimize the damage in case of chlorine gas leak. If walls are to be painted, they should be painted with paint which chlorine cannot attack, *i.e. epoxy*.

4.2.4.9. Weigh Scales

A platform scale of proper size shall be provided for weighing cylinders. For tonne cylinders indicating and recording type are recommended. Scales should be of corrosion resistant material and accessible, either by recessing or providing a ramp for convenience of moving the cylinder on and off the scale platform.

4.2.4.10. Lighting

Both the chlorination equipment room and the cylinder storage room should be installed with proper lighting fixtures to make the inside of these rooms visible from the observation windows. About 60 foot candles lighting intensity at the chlorinator and around the cylinders should be sufficient. Lights should be explosion proof.

4.2.4.11. Emergency Ventilation

Both the chlorination equipment room, the cylinder storage room and below ground pits, vaults, basements, etc. shall be provided with air evacuation fans or a similar mechanical exhaust system, which could be turned on, manually from a safe location (outside entry door) in case of an emergency.

Air inlets shall be located so as to provide cross ventilation with air, and such that temperatures will not adversely affect the chlorination equipment. The exhaust should be routed at least one-third the height of the building above the roof or if it is to be located on an exterior wall, it must not be located in the area of the exit door or fresh air inlet. The fresh air inlet should be located at the opposite end of the room from the exhaust outlet to facilitate complete air replacement.

Emergency mechanical ventilation should be of sufficient capacity to produce one complete air change per minute. The air intake duct shall extend to within 150 mm of the floor and shall be located closest to the most likely source of leakage.

For emergency gas leaks, the gas shall be exhausted to the outside, unless conditions dictate otherwise. The location of the exhaust vent should be determined carefully so that it does not pose a health hazard to people, or damage the environment outside the building.

Vents from feeders (e.g. gas regulators) shall discharge to the outside atmosphere, above grade and to a safe location where personnel will not be endangered from the escaping gas. Vents shall not under any circumstance be located in the vicinity of an exit door or the fresh air inlet.

4.2.4.12. Warning Devices

1. The chlorination equipment room, the cylinder storage room, and below ground pits, vaults, and basements must be equipped with an automatic gas detection and related alarm equipment. The detection system should activate an emergency light and sound an alarm system that is visible and audible within the facility.

2. The alarm should have a distinct sound to be easily noticeable within the facility.

3. The exterior alarm light is to be located in an area where it can be easily seen on approach to the building.

4. A strong solution of aqueous ammonia (18 Baume or higher) should be available for use in locating the source of chlorine leaks. Dense, white clouds of ammonium chloride are formed by the reaction of the ammonia and chlorine, thus confirming the source of the chlorine leak.

5. In facilities using 1 tonne cylinders, consideration should also be given to the provision of caustic soda solution reaction tanks for absorbing the contents of leaking cylinders.

4.2.4.13. Temperature Control

The chlorination equipment, the cylinder storage room and below ground pits, vaults, and basements shall be equipped with necessary heating and ventilation equipment to maintain the temperature within the recommended range. The rate at which the gas is withdrawn from the cylinders is temperature dependent and the withdrawal rate decreases at lower temperatures. Maintenance of temperature within a narrow range, 18 to 21°C, would ensure a constant rate of gas withdrawal from the cylinders.

4.2.4.14. Cross-connection Protection

The chlorinator water supply piping shall be designed to prevent contamination of the treated water supply by sources of questionable quality. At all facilities treating surface water, pre- and post-chlorination systems must be independent to prevent possible siphoning of partially treated water into the clear well or distribution system. The water supply to each injector shall have a separate shut-off valve. No master shut-off valve will be allowed.

4.2.4.15. Chlorine Piping

1. The piping conveying chlorine gas to the chlorinator should slope gently upwards to the chlorinator to facilitate the return of any condensate to the cylinders. This precaution is especially useful for pressurized lines.

2. The chlorine gas piping between the cylinders, chlorinators and injectors, etc., should not be located on an outside wall or in a location where low temperatures may be encountered.

3. All piping should be colour coded to clearly identify their functions (see Section 3.1.13.).

4. Piping systems should be as simple as possible, manufactured to be suitable for chlorine service and with a minimum number of joints.

5. The pipes carrying elemental liquid or dry gaseous chlorine under pressure must be Schedule 80 seamless steel tubing or other materials recommended by the Chlorine Institute (never use PVC). A minimum size of at least 19 mm diameter is recommended. Due to the corrosiveness of wet chlorine all lines designed to handle dry chlorine should be protected from the entrance of water or moist air. PVC tubing should not be employed for conveying chlorine from cylinders to the chlorinator unless the tubing is under negative pressure.

6. Nylon products are not acceptable for any part of the chlorine solution piping system.

4.2.4.16. Injector

1. Each injector must be selected for the point of application with particular attention given to the quantity of chlorine to be added, the maximum injection water flow, the total discharge back pressure, the injector operating pressure, and the size of chlorine solution line. Gauges for measuring water pressure and vacuum at the inlet and outlet of each injector should be provided.

2. The chlorine solution injector/diffuser must be compatible with the point of application to provide a rapid and thorough mix with all the water being treated. The centre of a pipeline is the preferred application point.

3. It is recommended that a strainer be installed on the waterline to the injector. This prevents any possible grit or foreign material from entering and blocking the injector, or causing undue wear on the injector throat and tailway.

4. If a booster pump is used in the system, the strainer should precede the pump. Also, two booster pumps shall be provided to ensure continuous operation of the chlorination system.

4.2.4.17. Methods of Dosage Control

4.2.4.17.1. Open Loop Flow Proportional Control

Automatic proportioned-to-flow control consists of varying the rate of chlorine feed in proportion to the flow as determined be a metering device. The dosage rate is manually set, and the control device varies the rate in relation to flowrate. The chlorinator may be either automatic or manual start and stop.

Usually over-chlorination is practised to ensure results. Invariably, such a device wastes some chlorine.

4.2.4.17.2. Closed-Loop Flow Proportional Control (Compound-Loop Arrangement with One Chlorine Analyzer)

Chlorine residual analyzer provides feedback to the chlorinator. Flow signal and dosage signal each separately control the added chlorine feed with a compound-loop arrangement. If the residual is above the pre-determined level, the chlorine feed rate is reduced, and vice versa. In some designs, chlorine residual is measured at one point in the system and in other designs at 2 or 3 points.

4.2.4.17.3. Closed-Loop Flow Proportional Control (Compound-Loop Arrangement with Two Chlorine Analyzers)

This ideal system employs quantitative as well as qualitative feed control as in the previous case. However, the qualitative control is accomplished at two points in the flow stream. One sample is automatically collected immediately downstream from the point of chlorination (diffuser) and analyzed by another chlorine analyzer, which monitors the combined residual after a given contact time and adjusts the control point on the analyzer, which controls the chlorine metering equipment. When the residual chlorine is more than the desired (pre-set) level, the chlorine feed rate is reduced and vice versa.

4.2.4.17.4. Required Chlorine Control Systems

Plants with proper qualitative and quantitative control systems are known to chlorinate effectively and efficiently. However, the plants without such controls show either inadequate performance (due to under dosage) or waste chlorine unnecessarily (by undue overdose). Higher than needed chlorine residuals may result in ecological damage to the receiving waters.

Table 4.6 summarizes chlorine control guidelines.

Table 4.6
Chlorine Control Guidelines

Size of Plant	Type of Receiving Water	Recommended Control	Method of Chlorine Residual Determination
Large	All Types	Closed-loop, Flow Proportional, Two Chlorine Analyzers	Amperometric Titrator; Continuous Determination and Recording
Medium	Ecologically Sensitive Waters with Fishing Potential	Same as Large Plants	Same as Large Plants
	Waters of Public Health Importance	Closed-loop, Flow Proportional, One Chlorine Analyzer	Amperometric Titrator; Continuous Determination (Optional Recording)
Small	Ecologically Sensitive Receiving Water	Closed-loop, Flow Proportional, One Chlorine Analyzer	Amperometric Titrator; Continuous Determination
	Receiving Water of Public Health Importance	Open-loop, Flow Proportional	Starch-iodide Method Orthotolidine Method (Intermittent – Manual)

4.2.4.18. Gas Protection and Safety Equipment

1. Each facility should have, readily available, a self-contained or air supplied breathing apparatus which works on pressure demand or constant flow principle. The equipment shall be stored in a convenient location, but not inside any room where chlorine is used or stored. The unit shall use compressed air, have at least 30-min capacity, and be compatible with the units used by the fire department responsible for the facility. The air cylinder shall be purged and refilled on a regular basis in order to ensure its safe operation. Operator(s) shall be trained in the use of the breathing apparatus.

2. Safety chains should be used to retain all cylinders, either in storage or on weigh scales, in a safe, upright position, except when the cylinders are stored horizontally in cradles specially built for their storage in accordance with the manufacturer's recommendations.

3. 1-tonne cylinders must be moved by an approved lifting bar and hoist, and not by rolling them along the floor.

4.2.4.19. Eye Wash Stations and Showers

1. For facilities using cylinders, eye wash stations should be located within 4.5 m outside of the chlorine equipment room and/or chlorine storage room exit doors. An eye wash station capable of providing a continuous 15-minute flush as a minimum, using potable water or equivalent, shall be installed in a convenient location(s) within the facility.

2. For facilities using 1-tonne cylinders, then showers shall be located within 4.5 m of both the chlorine equipment room or chlorine storage room exit doors.

3. A water holding tank that will allow water to come to room temperature should be installed in the water line feeding the shower and eye washing device. Other methods of water tempering will be considered on an individual basis.

4.2.4.20. Other Protection

1. Clothing, including gloves, goggles, safety shoes, etc. should be available for personnel handling chlorination equipment.

2. Other safety equipment should include a first aid kit and a fire extinguisher.

3. Container repair emergency kits, such as Chlorine Institute Emergency Kit A for 68 kg cylinders and Kit B for tonne containers, should be available at all chlorination facilities. A contingency plan to deal with major leaks should also be in place.

All the above equipment should be located at readily accessible points, away from the areas likely to be contaminated with chlorine gas. Instructions for using, testing and replacing parts should be posted near the equipment.

A summary of emergency procedures shall be posted outside the chlorine room or chlorine storage room.

4.2.4.21. Operation

For effective operation, chlorination equipment requires care and attention. Manufacturer’s recommendations and instructions should be made available and should be followed.

4.2.4.22. Fencing

The chlorination facility shall be located in a fenced enclosure to minimize vandalism and endangerment to the general public in case of a chlorine leak.

4.2.4.23. System Design and Installation

Chlorination systems should only be designed by registered professional engineers, and plans for the systems must be submitted to the DOEC for approval.

4.2.4.24. Deviation from these Guidelines

For design of facilities where the consultant considers these guidelines not to be applicable, the DOEC should be contacted directly.

4.2.5. Hypochlorination - Facility Design

4.2.5.1. Capacity

Chlorination equipment shall be sized to provide the specified free chlorine residual after 20 minutes of contact time at the maximum design flow rate. **It shall be designed to maintain proper chlorine residual under variable chlorine demand conditions, including peak flow.**

4.2.5.2. Standby Equipment

Where chlorination is required for protection of the supply, standby equipment of sufficient capacity should be available to replace the largest unit.

Spare parts and tools shall be made available to replace parts subject to wear and breakage. Examples of spare parts are rubber fittings, hose clamps, gaskets and glassware.

4.2.5.3. Application of Chlorine

Depending on the water system, the chlorination installation should have a chlorine residual analyzer with recorder to accurately record the chlorine residual continually. The recorder should be accurate, reliable and approved by the DOEC.

Chlorination equipment should be designed as to be able to feed the desired level of chlorine into the supply line on a flow proportional basis.

4.2.5.4. Automatic Proportioning

Where flow varies, an automatic flow proportional system should be installed. If chlorine demand varies then a residual analyzer with recorder should be installed. If both the flow and the chlorine demand vary, then a compound loop system should be installed.

4.2.5.5. Certification and Storage Requirements

All sodium and calcium hypochlorite must be NSF 60 (National Sanitation Foundation) certified to ensure it is suitable for use in potable water. The NSF 60 Water Treatment Program is responsible for the certification of drinking water treatment chemicals to ensure that these products do not contribute contaminants to drinking water that could cause adverse health effects.

The sodium hypochlorite shall be stored in a dark cool area to minimize loss of strength of the solution. The rate of strength loss for sodium hypochlorite doubles with every 5EC rise in temperature. In this regard a storage cabinet should be provided and shall be located away from any direct heat and light sources. Because of its shelf life consideration must be given when purchasing sodium hypochlorite, as too large quantities will be wasting money and possibly cause some impact on the adequacy of the chlorination.

Calcium hypochlorite (also known as HTH) should always be stored indoors and kept dry and covered. When it gets wet, chlorine gas can be released, and corrosive products formed. In this regard, there should not be any automatic sprinklers where it is stored. Calcium hypochlorite will not burn, but if heated above 175EC will release oxygen that will support a fire, therefore it should never be stored around combustible materials. It must also be kept away from direct contact with oil, grease or gasoline vapours. Direct contact can cause a fire.

4.2.5.6. Floor Drains

All drains shall be equipped with screens to prevent the entry of insects, birds and rodents.

4.2.5.7. Hose Bibbs and Cross-Connection Control

Most hypochlorination facilities will require a water line to supply water to the solution tank. Common practice is to use a hose bibb connection with a garden hose. In this regard a backflow prevention device, in this case, a non-removable hose connection vacuum breaker meeting or exceeding CSA 64.2, shall be attached to any hose bibb connection in order to reduce the possibility of contaminants or strong chlorine solutions entering the potable water system due to back-siphonage. The use of any other type of waterline must be protected with the appropriate backflow prevention device.

4.2.5.8. Lighting

The chlorination equipment room should be installed with proper lighting fixtures. About 60 foot candles lighting intensity at the chlorinator

4.2.5.9. Ventilation

Adequate ventilation should be provided in the facility, particularly where calcium hypochlorite is used in order to control the dust from the calcium hypochlorite, which can be a potential health hazard.

4.2.5.10. Temperature Control

The chlorination equipment room should be installed with necessary heating and ventilation equipment to maintain the temperature within the recommended range, not below 13EC and not above 27EC.

4.2.5.11. Chlorine Solution Tank and Piping

Because hypochlorite solutions are corrosive, solution tanks are usually made of polyethylene or fibreglass. The solution tank should have a tight fitting corrosion resistant cover, which helps to prevent the weakening of the solution, and possible contamination.

1. All pipes and tubes should be colour coded to clearly identify their functions.

2. Piping systems should be as simple as possible, manufactured to be suitable for hypochlorination and with a minimum number of joints. Polyethylene is normally used for hypochlorination solutions.

4.2.5.12. Point of Application

1. The point of application must provide a rapid and thorough mix with all the water being treated. The centre of a pipeline is the preferred application point. A diffuser can be added at the end of the discharge line to assist in this regard. The total backpressure in the water line must also be considered.

4.2.5.13. Methods of Dosage

Refer to Section 4.2.4.17.

4.2.5.14. Eye Wash Stations

An eye wash station capable of providing a continuous 15-minute flush as a minimum, using potable water or equivalent, shall be installed in a convenient location(s) within the facility.

A water holding tank that will allow water to come to room temperature must be installed in the water line feeding the shower and eye washing device. Other methods of water tempering will be considered on an individual basis.

4.2.5.15. Other Protection

1. Clothing, including gloves, goggles, safety shoes, etc. should be available for personnel handling chlorination equipment.

2. Other safety equipment should include a first aid kit and a fire extinguisher.

All the above equipment should be located at readily accessible points. Instructions for using, testing and replacing parts should be posted near the equipment.

4.2.5.16. Operation

For effective operation, chlorination equipment requires care and attention. Manufacturer’s recommendations and instructions should be made available and should be followed.

4.2.5.17. Fencing

The chlorination facility shall be located in a fenced enclosure to minimize vandalism.

4.2.5.18. System Design and Installation

Chlorination systems should only be designed by registered professional engineers, and plans for the systems must be submitted to the DOEC for approval.

4.2.5.19. Deviation from these Guidelines

For design of facilities where the consultant considers these guidelines not to be applicable, the DOEC should be contacted directly.

4.3. Ozone

Ozonation uses the same form of ozone found in the atmosphere. Ozone must be generated at the point of use, as it is highly unstable and can't be stored as a compressed gas. It is formed by passing dry clean air between two high voltage electrodes. Oxygen gas can be used and will produce approximately twice the amount of ozone at the same electrical input. Ozone is a powerful oxidant over a wide pH and temperature range. At dosage concentrations of 1.0 to 1.5 mg/L it will remove colour, taste and odours from drinking water and will inactivate disease-causing microbes including *Giardia* and *Cryptosporidium*.

Unlike chlorine, ozone disinfection dissipates quickly in water supplies. Contaminants entering an ozonated water supply after treatment has occurred will be left unaffected. Adding chlorine before treated water enters the distribution system provides residual protection straight to the tap.

Advantages

1. Ozone is more effective than chlorine in destroying viruses and bacteria;
2. Ozone is a good oxidant for removing colour;
3. Ozone is an effective oxidizer for removal of iron and manganese;
4. Ozone can improve performance of subsequent coagulation, settling, and filtration;
5. Ozonation is an effective way to remove taste and odour;
6. Ozone is recognized as an important tool in controlling halogenated disinfection by-products;
7. There is no re-growth of microorganisms after ozonation, unlike ultraviolet and chlorine disinfection; and
8. Ozone is generated onsite, and thus, there are fewer safety problems associated with shipping and handling.

Disadvantages

1. Low dosages may not effectively inactivate some viruses, spores, and cysts;
2. Ozonation is more complex than other disinfection technologies. Water treatment plant operators require additional training to operate ozonation treatment equipment;

3. Ozone is very reactive and corrosive, thus requiring corrosion-resistant material, such as stainless steel. Ozonation can also lead to increased rates of corrosion in the distribution system, requiring more frequent replacement of infrastructure;
4. Ozonation is not economical for poor quality and poorly treated water. Because of the massive amount of electricity necessary for treatment, the cost of ozonation is approximately 4 times larger than that of traditional chlorine disinfection. The cost of treatment is relatively high, being both capital and power intensive;
5. Ozone is extremely irritating and possibly toxic, so off-gases from the contactor must be destroyed to prevent worker exposure;
6. Although ozonation does not produce the harmful by-products associated with chlorine disinfection, ozonation will create bromate, a known carcinogen, if bromide is present in the water supply; and
7. There is no measurable residual to indicate the efficacy of ozone disinfection.

4.3.1. Ozone Generator

4.3.1.1. Capacity

1. The production rating of the ozone generators shall be stated in kg/day and kW-hr/kg at a maximum cooling water temperature and maximum ozone concentration.

2. The design shall ensure that the minimum concentration of ozone in the generator exit gas will not be less than 1.0%, by weight.

3. Generators shall be sized to have sufficient reserve capacity so that the system does not operate at peak capacity for extended periods of time, which can result in premature breakdown of the dielectrics.

4. The production rate of ozone generators will decrease with a variation in the supply temperature of the coolant throughout the year. Curves or other data shall be used to determine production changes due to the temperature change of the supplied coolant. The design shall ensure that the generators can produce the required ozone at maximum temperature.

5. Appropriate ozone generator backup equipment must be provided.

4.3.1.2. Electrical

The generators can be low, medium or high frequency type. Specifications shall require that the transformers, electronic circuitry and other electrical hardware be proven, high quality components designed for ozone service.

4.3.1.3. Cooling

The required water flow to an ozone generator varies with the ozone production. Normally unit design provides a maximum cooling water temperature rise of 2.8°C (5°F). The cooling water must be properly treated to minimize corrosion, scaling and microbiological fouling of the water side of the tubes. A closed loop cooling water system is often used to ensure proper water conditions are maintained. Where cooling water is treated, cross connection control shall be provided to prevent contamination of the potable water supply.

4.3.1.4. Materials

To prevent corrosion, the ozone generator shell and tubes shall be constructed of Type 316L stainless steel.

4.3.2. Ozone Contactors

Analyses for bromate production should be done early in the design process. Biological mediation or other by-product removal processes shall be included as a part of the ozone facility design.

The selection or design of the contactors and method of ozone application depends on the purpose for which the ozone is being used.

1. Bubble Diffusers. Where disinfection is the primary application, a minimum of two contact chambers, each equipped with baffles to prevent short-circuiting and induce counter current flow, shall be provided. Ozone shall be applied using porous-tube or dome diffusers.

2. The minimum contact time shall be 10 minutes. A shorter contact time may be approved if justified by appropriate design and CT considerations.

3. For ozone applications in which precipitates are formed, such as with iron and manganese removal, porous diffusers should be used with caution.

4. Where taste and odour control is of concern, multiple application points and contactors shall be considered.

5. Contactors should separate closed vessels that have no common walls with adjacent rooms. The contactors must be kept under negative pressure and sufficient ozone monitors shall be provided to protect worker safety. Placement of the contactors where the entire roof is exposed to the open atmosphere is recommended. In no case shall the contactor roof be a common wall with a separate room above the contactors.

6. Large contact vessels should be made of reinforced concrete. All reinforcement bars shall be covered with a minimum of 4.0 cm of concrete. Smaller contact vessels can be made of stainless steel, fibreglass or other material which will be stable in the presence of residual ozone and ozone in the gas phase above the water level.

7. Where necessary, a system shall be provided between the contactors and the off-gas destruction unit to remove froth from the air and return the other to the contactors or other location acceptable to the DOEC. If foaming is expected to be excessive, then a potable water spray system shall be placed in the contactors headspace.

8. All openings into the contactors for pipe connections, hatchways, etc. shall be properly sealed using welds or ozone resistant gaskets such as Teflon or Hypalon.

9. Multiple sampling ports shall be provided to enable sampling of effluent from each compartment and to confirm CT calculations.

10. A pressure/vacuum relief valve shall be provided in the contactors and piped to a location where there will be no damage to the destruction unit.

11. The diffusion system should work on a counter current basis such that the ozone is fed at the bottom of the vessel and water is fed at the top of the vessel.

12. The depth of water in bubble diffuser contactors should be a minimum of 6 m. The contactors should have a minimum of 1 m of freeboard to allow for foaming.

13. All contactors shall have provisions for cleaning, maintenance and drainage of the contactors. Each contactor compartment shall be equipped with an access hatchway.

14. Aeration diffusers shall be fully serviceable by either cleaning or replacement.

15. Other contactors, such as the venturi or aspirating turbine mixer contactors, may be approved by the DOEC provided adequate ozone transfer is achieved and the required contact times and residuals can be verified.

4.3.3. Ozone Destruction Unit

A system for treating the final off-gas from each contactor must be provided in order to meet safety and air quality standards. Acceptable systems include thermal destruction and thermal/catalytic destruction units. In order to reduce the risk of fires, the use of units that operate at lower temperature is encouraged, especially where high purity oxygen is the feed gas. The maximum allowable ozone concentration in the discharge is 0.1 ppm (by volume). At least two units shall be provided which are each capable of handling the entire gas flow. Exhaust blowers shall be provided in order to draw off-gas from the contactors into the destruction unit. Catalysts must be protected from froth, moisture, and other impurities that may harm the catalyst. The catalyst and heating elements shall be located where they can easily be reached for maintenance.

4.3.4. Piping Materials

Only low carbon 304L and 316L stainless steels shall be used for ozone service with 316L the preferred.

4.3.5. Joints and Connections

Connections on piping used for ozone service are to be welded where possible. Connections with meters, valves, or other equipment are to be made with flanged joints with ozone resistant gaskets, such as Teflon or Hypalon. Screwed fittings shall not be used because of their tendency to leak. A positive closing plug or butterfly valve plus a leak-proof check valve shall be provided in the piping between the generator and the contactors to prevent moisture reaching the generator.

4.3.6. Instrumentation

1. Pressure gauges shall be provided at the discharge from the air compressor, at the inlet to the refrigeration dryers, at the inlet and outlet of the desiccant dryers, at the inlet of the ozone generators and contactors and at the inlet to the ozone destruction unit.

2. Electric power meters should be provided for measuring the electric power supplied to the ozone generators. Each generator shall have a trip, which shuts down the generator when the wattage exceeds a certain preset level.

3. Dew point monitors shall be provided for measuring the moisture of the feed gas from the desiccant dryers. Because it is critical to maintain the specified dew point, it is recommended that continuous recording charts be used for dew point monitoring which will allow for proper adjustment of the dryer cycle. Where there is potential for moisture entering the ozone generator from downstream of the unit or where moisture accumulation can occur in the generator during shutdown, post-generator dew point monitors shall be used.

4. Airflow meters shall be provided for measuring airflow from the desiccant dryers to each of other ozone generators, air flow to each contactor and purge airflow to the desiccant dryers.

5. Temperature gauges shall be provided for the inlet and outlet of the ozone cooling water and the inlet and outlet of the ozone generators feed gas, and, if necessary, for the inlet and outlet of the ozone power supply cooling water.

6. Water flow meters shall be installed to monitor the flow of cooling water to the ozone generators and, if necessary, to the ozone power supply.

7. Ozone monitors shall be installed to measure ozone concentration in both the feed-gas and off-gas from the contactors and in the off-gas from the destruction unit. For disinfection systems, monitors shall also be provided for monitoring ozone residuals in the water. The number and location of ozone residual monitors shall be such that the amount of time that the water is in contact with the ozone residual can be determined.

8. A minimum of one ambient ozone monitor shall be installed in the vicinity of the contactors and a minimum of one ambient ozone monitor shall be installed in the vicinity of the generator. Ozone monitors shall also be installed in any areas where ozone gas may accumulate.

4.3.7. Alarms

The alarm/shutdown systems listed here should be considered at each installation:

1. Dew point shutdown/alarm. This system should shut down the generator in the event the system dew point exceeds -60°C (-76°F).

2. Ozone generator cooling water flow shutdown/alarm. This system should shut down the generator in the event that cooling water flows decrease to the point that generator damage could occur.

3. Ozone power supply cooling water flow shutdown/alarm. This system should shut down the power supply in the event that cooling water flow decreases to the point that damage could occur to the power supply.

4. Ozone generator cooling water temperature shutdown/alarm. This system should shutdown the generator if either the inlet or outlet cooling water exceeds a certain preset temperature.

5. Ozone power supply cooling water temperature shutdown/alarm. This system should shutdown the power supply if either the inlet or outlet cooling water exceeds a certain preset temperature.

6. Ozone generator inlet feed-gas temperature shutdown/alarm. This system should shutdown the generator if the feed-gas temperature is above a preset value.

7. Ambient ozone concentration shutdown/alarm. The alarm should sound when the ozone level in the ambient air exceeds 0.1 ppm or a lower value chosen by the water supplier. Ozone generator shutdown should occur when ambient ozone levels exceed 0.3 ppm (or a lower value) in either the vicinity of the ozone generator or the contactor.

8. Ozone destruct temperature alarm. The alarm should sound when temperature exceeds a preset value.

4.3.8. Safety

The maximum allowable ozone concentration in the air to which workers may be exposed must not exceed 0.1 ppm (by volume). Noise levels resulting from the operating equipment of the ozonation system shall be controlled to within acceptable limits by special room construction and equipment isolation. High voltage and high frequency electrical equipment must meet current electrical and fire codes. Emergency exhaust fans must be provided in the rooms containing the ozone generators to remove ozone gas if leakage occurs. A portable purge air blower that will remove residual ozone in the contactors prior to entry for repair or maintenance should be provided.

4.3.9. Construction Considerations

Prior to connecting the piping from the desiccant dryers to the ozone generators the air compressors should be used to blow the dust out of the desiccant. The contactors should be tested for leakage after sealing the exterior. This can be done by pressurizing the contactors and checking for pressure losses. Connections on the ozone service line should be tested for leakage using the soap-test method.

4.3.10. Ozone Feed Gas Preparation

Feed gas can be air, high purity oxygen, or oxygen enriched air. Air handling equipment on conventional low-pressure air feed systems shall consist of an air compressor, water/air separator, refrigerant dryer, and heat reactivated desiccant dryer, and particulate filters. Some “package” ozonation systems for small systems may work effectively operating at high pressure without the refrigerant dryer and with a “heat-less” desiccant dryer. In all cases the design engineer must ensure that the maximum dew point of -60°C (-76º F) will not be exceeded at any time. For oxygen- feed systems, dryers typically are not required.

4.3.10.1. Air Compression

1. Air compressors shall be of the liquid-ring or rotary lobe, oil-less positive displacement type for smaller systems or dry rotary screw compressors for larger systems.

2. The air compressors shall have the capacity to simultaneously provide for maximum ozone demand, provide the airflow required for purging the desiccant dryers (where required) and allow for standby capacity.

3. Air feed for the compressors shall be drawn from a point protected from rain, condensation, mist, fog, and contaminated air sources to minimize moisture and hydrocarbon content of the air supply.

4. A compressed air after-cooler and/or entrainment separator with automatic drain shall be provided prior to the dryers to reduce the water vapour.

5. A back-up air compressor must be provided so that ozone generation is not interrupted in the event of a breakdown.

4.3.10.2. Air Drying

1. Dry, dust free and oil- free feed gas must be provided to the ozone generator. Dry gas is essential to prevent formation of nitric acid, to increase the efficiency of ozone generation, and to prevent damage to the generator dielectrics. Sufficient drying to maximum dew point of –60ºC (-76º F) must be provided at the end of the drying cycle.

2. Drying for high-pressure systems may be accomplished using heatless desiccant dryers only. For low-pressure system, a refrigeration air dryer in series with heat-reactivated desiccant dryer shall be used.

3. A refrigeration dryer capable of reducing inlet air temperature to 4ºC (40ºF) shall be provided for low-pressure air preparation systems. The dryer can be of the compressed refrigerant type or chilled water type.

4. For heat-reactivated desiccant dryers, the unit shall contain two desiccant filled towers complete with pressure relief valves, two four- way valves and a heater. In addition, external type dryers shall have a cooler unit and blowers. The size of the unit shall be such that the specified dew point will be achieved during a minimum adsorption style time of 16 hours while operating at the maximum expected moisture loading conditions.

5. Multiple air dryers shall be provided so that the ozone generation is not interrupted in the event of dryer breakdown.

6. Each dryer shall be capable of venting "dry" gas to the atmosphere, prior to the ozone generator, to allow start-up when other dryers are "on-line."

4.3.10.3. Air Filters

1. Air filters shall be provided on the suction side of the air compressors, between the air compressors and the dryers and between the dryers and the ozone generators.

2. The filter before the desiccant dryers shall be of the coalescing type and be capable of removing aerosol and particulate larger than 0.3 microns in diameter. The filter after the desiccant dryer shall be of the particulate type and be capable of removing all particulate greater than 0.1 microns in diameter, or smaller if specified by the generator manufacturer.

4.3.10.4. Air Preparation Piping

Piping in the air preparation system can be common grade steel, seamless copper, stainless steel or galvanized steel. The piping must be designed to withstand the maximum pressures in the air preparation system.

4.4. UV

Ultraviolet irradiation is a process for inactivating microorganisms by irradiating them with ultraviolet light. The UV disinfection process takes place as water flows through an irradiation chamber. Microorganisms in the water are inactivated when the UV light is absorbed. A photochemical effect is created and vital processes are stopped within the cells, thus making the microorganisms harmless. UV light inactivates microbes by damaging their nucleic acid, thereby preventing the microbe from replicating. When a microbe cannot replicate, it is incapable of infecting a host. The ultraviolet light does not leave a disinfectant residual so a form of chlorine disinfection must be applied if a residual is desired. To allow the irradiation to reach the organisms effectively, the water to be disinfected must be relatively free of particles, as in filtered water.

Advantages
1. The UV light disinfection process does not use chemicals;
2. Microorganisms, including bacteria, viruses, and algae, are inactivated within seconds of UV light disinfection, but all are not equally sensitive. Generally, viruses and algae are more resistant to disinfection by UV light.
3. UV light is effective in inactivating *Cryptosporidium*, while at the same time decreasing chlorinated disinfection by-products;
4. UV disinfection is used in air and water purification, sewage treatment, protection of food and beverages, and many other disinfection and sterilization processes; and
5. One major advantage of UV light disinfection is that it is capable of disinfecting water faster than chlorine, and without the need for retention tanks or potentially harmful chemicals.

Disadvantages
1. Scaling of lamps can be an issue if dissolved organics are present and will gradually decrease the effectiveness of the UV system;
2. Turbidity or the presence of suspended particles can prevent or shadow bacteria from the UV dose. Problems typically exist in surface water applications and filtration may be required to remove turbidity to have an effective system. NSF studies have shown that the effective transmission coefficient of the water should be 75%;
3. UV may be used as a primary disinfection process but will need to be followed by a secondary disinfection process to maintain a measurable residual in the distribution network;
4. Capital and operational costs will be significantly higher than chlorine disinfection alone; and
5. Effective lamp life and replacement will require additional operational requirements in addition to power requirements.

4.4.1. UV Reactor Design

All UV disinfection facilities must continuously monitor such parameters that allow the operator to determine that the target design 254nm-equivalent UV pass through dose or higher is being delivered, and all systems must provide failure alarms when this design dose is not being delivered.

Equipment that records test results of the continuous monitoring equipment is strongly recommended for drinking water systems using a surface water supply or a groundwater supply under the direct influence of surface water. All sensors that constitute part of the monitoring system must be calibrated at a frequency that maintains their necessary sensitivity and reliability in ensuring that the design UV dose is being achieved, or as deemed necessary by the DOEC.

Most conventional UV reactors are available in two types; closed vessel and open channel. For drinking water applications, the closed vessel is generally the preferred UV reactor for the following reasons:
1. Smaller footprint;
2. Minimized pollution from airborne material;
3. Minimal personnel exposure to UV; and
4. Modular design for installation simplicity.

Additional design features for conventional UV disinfection systems include:
1. UV sensors to detect any drop in UV lamp output intensity;
2. Alarms and shut-down systems;
3. Automatic or manual cleaning cycles; and
4. Telemetry systems for remote installations.

In addition to conventional UV systems, two other UV processes are currently being evaluated for drinking water disinfection; micro-screening/UV, and pulsed UV. Both of these systems profess to provide sufficient UV dose to inactivate *Giardia* cysts and *Cryptosporidium* oocysts.

The most common point of application for UV radiation is the last step in the treatment process train just prior to the distribution system and after filtration. The use of UV disinfection has no impact on other processes at the water treatment facility.

4.4.1.1. Hydraulic Design Considerations

The major elements that should be considered in the hydraulic design of a UV closed vessel reactor are:
Dispersion;
Turbulence;
Effective volume;
Residence time disturbance; and
Flowrate.

Dispersion is the characteristic of water elements to scatter spatially. The ideal UV reactor is plug flow, where water particles are assumed to discharge from the reactor in the same sequence they entered and each element of water passing through the reactor resides in the reactor for the same period of time. An ideal plug flow reactor has no dispersion and is approximated by a long tank with high length-to-width ratio in which dispersion is minimal.

In addition to plug flow characteristics, the ideal UV reactor has a flow that is turbulent radially from the direction of flow, to eliminate dead zones. This radially turbulent flow pattern promotes uniform application of UV radiation. A negative of having a radially turbulent flow pattern is that some axial dispersion results, thus disrupting the plug flow characteristics. Techniques such as misaligning the inlet and outlet, and using perforated stilling plates, have been used to accommodate the contradicting characteristics of plug flow and turbulence.

4.4.2. Pathogen Inactivation

UV radiation is efficient at inactivating vegetative and sporous forms of bacteria, viruses, and other pathogenic microorganisms. Electromagnetic radiation in the wavelengths ranging from 240 to 280 nm effectively inactivates microorganisms by irreparably damaging their nucleic acid. The most potent wavelength for damaging deoxyribonucleic acid (DNA) is approximately 254 nm (Wolfe, 1990).

The germicidal effects of UV light involve photochemical damage to RNA and DNA within the microorganisms. Microorganism nucleic acids are the most important absorbers of light energy in the wavelength of 240 to 280 nm (Jagger, 1967). DNA and ribonucleic acid (RNA) carry genetic information necessary for reproduction; therefore, damage to either of these substances can effectively sterilize the organism. Damage often results from the dimerization of pyrimidine molecules. Replication of the nucleic acid becomes very difficult once the pyrimidine molecules are bonded together due to the distortion of the DNA helical structure by UV radiation (Snider et al., 1991). Moreover, if replication does occur, mutant cells that are unable to replicate will be produced (USEPA, 1996).

Two phenomena of key importance when using UV disinfection in water treatment are the dark repair mechanisms and the capability of certain organisms to photoreactivate following exposure to certain light wavelengths. Under certain conditions, some organisms are capable of repairing damaged DNA and reverting back to an active state in which reproduction is again possible. The extent of reactivation varies among organisms. Because DNA damage tends to become irreversible over time, there is a critical period during which photoreactivation can occur. To minimize the effect of photoreactivation, UV contactors should be designed to either shield the process stream or limit the exposure of the disinfected water to sunlight immediately following disinfection.

4.4.2.1. Bacteria and Virus Inactivation

UV doses required for bacteria and virus inactivation are relatively low. One study determined that UV was comparable to chlorination for inactivation of heterotrophic plate count bacteria following treatment using granular activated carbon (Kruithof et al., 1989).

4.4.2.2. Protozoa Inactivation

Even though protozoa were once considered resistant to UV radiation, recent studies have shown that ultraviolet light is capable of inactivating protozoan parasites. However, results indicate that these organisms require a much higher dose than that needed to inactivate other pathogens.

4.4.3. Disinfection Efficiency

To achieve inactivation, UV should be absorbed into the microorganism. Therefore, anything that prevents UV from reacting with the microorganism will decrease the disinfection efficiency. Several factors that are known to affect disinfection efficiency of UV are:

1. Chemical and biological films that develop on the surface of UV lamps;
2. Dissolved organics and inorganics;
3. Clumping or aggregation of microorganisms;
4. Turbidity;
5. Colour; and
6. Short-circuiting in water flowing through the UV contactor.

4.4.3.1. Chemical Films and Dissolved Organics and Inorganics

Accumulation of solids onto the surface of the UV sleeves can reduce the applied UV intensity and, consequently, disinfection efficiency. In addition to biofilms caused by organic material, build-up of calcium, magnesium, and iron scales have been reported (DeMers and Renner, 1992). Waters containing high concentrations of iron, hardness, hydrogen sulphide, and organics are more susceptible to scaling or plating, which gradually decreases the applied UV intensity. Scaling is likely to occur if dissolved organics are present and inorganic concentrations exceed the following limits (DeMers and Renner, 1992):

1. Iron greater than 0.1 mg/L;
2. Hardness greater than 140 mg/L; and
3. Hydrogen sulphide greater than 0.2 mg/L.

4.4.3.2. Microorganism Clumping and Turbidity

Particles can affect the disinfection efficiency of UV by harbouring bacteria and other pathogens, partially protecting them from UV radiation, and scattering UV light. Typically, the low turbidity of ground water results in minimal impact on disinfection efficiency. However, the higher turbidity of surface water can impact disinfection efficiency. Similar to particles that cause turbidity, microorganism aggregation can impact disinfection efficiency by harbouring pathogens within the aggregates and shade pathogens that would otherwise be inactivated.

4.4.3.3. Reactor Geometry and Short Circuiting

Poor geometry within the UV contactor (which creates spacing between lamps) can leave dead areas where inadequate disinfection occurs (Hazen and Sawyer, 1992). A key consideration to improving disinfection is to minimize the amount of dead spaces where limited UV exposure can occur. Plug flow conditions should be maintained in the contactor; however, some turbulence should be created between the lamps to provide radial mixing of flow. In this manner, flow can be uniformly distributed through the varying regions of UV intensity, allowing exposure to the full range of available UV radiation (Hazen and Sawyer, 1992). As mentioned earlier, UV systems typically provide contact times on the order of seconds. Therefore, it is extremely important that the system configuration limit the extent of short-circuiting.

4.4.4. Operational Considerations

Onsite pilot plant testing is recommended to determine the efficiency and adequacy of UV disinfection for a specific quality of water. The efficiency test should involve injecting select microorganisms into influent water and sampling effluent water to determine survival rates. The National Sanitation Foundation's Standard 55 for ultraviolet water treatment systems recommends that UV disinfection systems not be used if the UV transmittance is less than 75% (NSF, 1991). If the raw water UV transmittance is less that 75%, the UV system should be preceded by other treatment processes (to increase UV transmittance) or a different disinfectant should be used. Some constituents that adversely interfere with UV disinfection performance by either scattering and/or absorbing radiation are iron, chromium, copper, cobalt, sulphites, and nitrites. Care should be taken with chemical processes upstream of UV disinfection process to minimize increasing concentrations of these constituents since disinfection efficiency may be adversely affected.

4.4.4.1. Equipment Operation

UV disinfection facilities should be designed to provide flexibility in handling varying flow rates. For lower flow rates, a single reactor vessel should be capable of handling the entire flow rate. A second reactor vessel with equal capacity of the first reactor vessel should be provided for redundancy should the first reactor vessel be taken out of service. For higher flow rates, multiple reactor vessels should be provided with lead/lag operation and flow split capacity to balance run time for each reactor vessel, if desired, and to avoid hydraulic overloading. Valves should be provided within the interconnecting piping to isolate one reactor vessel from another. There should also be a positive drainage system to remove water from within a reactor vessel when it is taken out of service.

4.4.4.1.1. UV Lamp Aging

The output of UV lamps diminishes with time. Two factors that affect their performance are:

1. Solarization, which is the effect UV radiation has on the UV lamp that causes it to become opaque; and

2. Electrode failure, which occurs when electrodes deteriorate progressively each time the UV lamp is cycled on and off.

Frequent lamp cycling will lead to premature lamp aging. When determining the requirement for UV disinfection, a 30% reduction of UV output should be used to estimate end of lamp. Average life expectancy for low-pressure UV lamps is approximately 8,800 hours.

4.4.4.1.2. Quartz Sleeve Fouling

Fouling of the quartz sleeve reduces the amount of UV radiation reaching the water. The quartz sleeve has a transmissibility of over 90% when new and clean. Over time, the surface of the quartz sleeve that is in contact with the water starts collecting organic and inorganic debris (e.g., iron, calcium, silt) causing a reduction in transmissibility (USEPA, 1996). When determining the requirements for UV disinfection, a 30% reduction of UV transmission should be used to reflect the effect of quartz sleeve fouling.

4.4.4.2. Equipment Maintenance

4.4.4.2.1. UV Lamp Replacement

Adequate space should be provided around the perimeter of the reactor vessels to allow access for maintenance and replacement of UV lamps. With modular electrical fittings, lamp replacement consists of unplugging the pronged connection of the old lamp and plugging in the new.

4.4.4.2.2. Quartz Sleeve Cleaning

Quartz sleeve cleaning may be accomplished by physical or chemical means. Physical alternatives include:

1. Automatic mechanical wiper;
2. Ultrasonic devices;
3. High water pressure wash; and
4. Air scour.

Chemical cleaning agents include sulphuric or hydrochloric acid. A UV reactor vessel may contain one or more physical cleaning system with provision for an occasional chemical cleaning.

4.4.4.2.3. Miscellaneous

Effective maintenance of a UV system will involve:
Periodic checks for proper operation;
Calibration of intensity meter for proper sensitivity; and
Inspect and/or clean reactor vessel interior.

4.5. Chlorine Dioxide

Chlorine dioxide has been generally recognized as a treatment for tastes caused by industrial wastes, such as phenols. However, chlorine dioxide can be used in the treatment of any taste and odour that is treatable by an oxidizing compound. Chlorine dioxide disinfects by oxidation, not substitution as with chlorine. The molecule oxidizes other compounds and forms the chlorite ion, which can subsequently reduce to chlorate and chloride.

Advantages

1. Chlorine dioxide is more effective than chlorine and chloramine for inactivation of viruses and protozoan;
2. Oxidizes iron, manganese and sulphides;
3. Can enhance clarification process;
4. Control of taste and odour problems from algae and decaying plant material can be achieved;
5. Halogenated by-products - THM formation is prevented as long as the generation system does not allow for the release of free chlorine;
6. Biocidal effectiveness is not affected by pH; and
7. Chlorine dioxide provides residual disinfection and can be used as a primary disinfectant.

Disadvantages

1. Chlorine dioxide forms DBPs of chlorite and chlorate;
2. Some generation technologies are difficult to maintain optimum yield and prevent excess chlorine from escaping unreacted;
3. Costs associated with chlorite, chlorate and training can be high;
4. Chlorine dioxide must be generated on site;
5. Chlorine dioxide decomposes in sunlight;

6. Extended storage of chlorine dioxide solution can contribute to by-product formation; and
7. Chlorine dioxide can produce noxious odours in some systems.

4.5.1. Safety Issues

Firstly, there are safety issues associated with chlorine dioxide as a gas. Chlorine dioxide can be an explosive gas at concentrations above 10% by volume. Due to its instability, it must be generated on site. The generator chosen must be designed in such a way that gaseous chlorine dioxide cannot accumulate. Most generation systems are designed with safety features, which will automatically shut the system down if catastrophic failures occur, such as the loss of dilution water.

The construction of a chlorine dioxide generator is not difficult. Commercially available generators are relatively simple, with pumps to move the precursors. For these types of systems, the safety aspects of the design have to be built in. That is, various controls are installed which are designed to work in case of a problem.

More elegant designs include those systems which are eductor driven. Such systems are relatively safe. Loss of dilution water leads to loss of vacuum and the generation of ClO_2 is interrupted. These systems rely on simple check valves to prevent the backflow of precursors. The system should be designed so that the concentration of chlorine dioxide does not exceed its solubility limit at any point in the system.

The second safety aspect is the storage of the precursors on site. For those plants desiring to eliminate gaseous chlorine storage, a three chemical feed approach would be appropriate. Storage of precursors can be done safely. However, of all the incidents which have arisen out of the use of chlorine dioxide, the largest number have been a result of improper storage or handling of the sodium chlorite solution. Attention to proper equipment selection and installation would solve these problems.

The third safety aspect is the operation of equipment by plant operators. Although modern generators can be made with proper safeguards, some knowledge of the generation process is required. An operator who operates the generator part time, and then only intermittently, will not be as alert to problems as compared to one who operates the unit as his/her primary responsibility.

4.5.2. Disinfection

When using chlorine dioxide for disinfection purposes, consideration must be given to the overall demand and should account for the following parameters:
1. Seasonal;
2. Variability;
3. Temperature; and
4. Application point.

For chlorine dioxide, pH has no effect on disinfection ability, unlike chlorine. Studies have demonstrated variations on the effectiveness of chlorine dioxide, but to what degree is still uncertain. Studies have shown that as pH increases the relative strength against *Giardia*, C*ryptosporidium* and viruses increases.

Disinfection capability decreases with a decrease of temperature. Changes in temperature to below 4°C can have a great affect on the ability to disinfect as at this point chlorine dioxide exists as a dissolved liquid and diffusion through the fluid is slower.

Suspended and colloidal materials greatly affect the disinfection ability of chlorine dioxide. Particles can hide bacteria from disinfection. It can increase the required CT by a factor of 3 depending on the microorganisms and the actual turbidity.

4.5.3. Taste and Odour Control

A common application for chlorine dioxide is to reduce the taste and odours present as a result of decaying plant material (algae). It is also capable of controlling the phenolic compounds that contribute to taste and odour problems. Taste and odour application points can typically be made after initial sedimentation or at the beginning of the plant where turbidity is low (< 10 NTU).

4.5.4. Oxidation of Iron and Manganese

Chlorine dioxide reduces to chlorite when oxidizing iron and manganese.

1. Iron oxidation - 1.2 mg/mg iron;
2. Manganese oxidation - 2.5 mg/mg manganese; and
3. Sulphite reduction - 5.8 mg/mg H_2S

4.5.5. Dosage Requirements

The following are the typical chlorine dioxide dosages that are required for various treatment methods:
Iron removal - 1.2 mg/mg iron – immediate;
Manganese removal - 2.5 mg/mg manganese – immediate;
Sulphide removal - 5.8 mg/mg H_2S;
Taste and odour - 1 - 2.5mg/L 10 minutes contact time;
Bacteria inactivation - 1 to 5 mg/L 5 minutes contact time;
Giardia - 1.5 to 2.0 ppm 60 minutes contact time; and
Cryptosporidium - approximately 8-16 times *Giardia*.

4.5.6. CT Value Enhancement

Chlorine dioxide has been studied extensively to provide CT values for viral, bacterial and protozoan removal. CT studies have shown proof that chlorine dioxide can be used to increase the credit given for removal of these pathogens.

4.5.7. On-site Generation

Chlorine dioxide cannot be stored for long periods of time and its reactivity when concentrated does not allow it to be transported. Generation on-site occurs through one of the following methods:

1. Acid/chlorite combination - two chemicals;
2. Chlorine gas method – three chemicals; and
3. Electrolytic membrane system - one chemical.

4.5.8. Disinfection By-products

The following items should be considered regarding chlorine dioxide and DBP formation:

1. Initial dosage/oxidation demand;

2. Blending ratios of sodium chlorite and chlorine in generation;

3. Exposure of chlorine dioxide to sunlight;

4. Reaction between chlorine and chlorite, if chlorine is used as a secondary disinfectant; and

5. Levels of chlorate in feedstock solutions.

Incomplete reactions in chlorine-based technologies can lead to chlorinated DBPs and chlorate as a result of the reaction between chlorite and chlorine. Interaction/oxidation of biological material increases overall demand and as a result increases DBP formation potential.

Generation methods that require subsequent storage of chlorine dioxide also produce DBPs in the form of chlorite and chlorate under influence of sunlight. Typically 50-70% of all chlorine dioxide applied to the water for disinfection will be reduced to chlorite, the remaining 30% will reduce to chlorate by means of other reactions with chlorine, pH dependence, and sunlight.

DBPs must be monitored on a basis determined by the exceedance of set guideline limits. Chlorine dioxide levels must also be monitored in the water leaving the plant by prescribed methodologies.

Typically, 70% of the initial dose of chlorine dioxide is converted to chlorite. Therefore if initial demand and dosage is kept below 1.4 ppm, then no DBP formation over the set guidelines will be experienced. Guideline for chlorite formation is 1.0 ppm. If a system does have a problem with chlorite formation, the level of chlorite in the system can be reduced using the following methods:

1. GAC or PAC application;
2. Adding reducing salts; ferrous chloride and ferrous sulphate; or
3. Adding reduced sulphur compounds; sulphur dioxide or sodium sulphite.

Option #3 is not prescribed as the process is effective at lower pH levels, thus the formation of chlorate can be a problem.

4.6. Chloramines

Chloramines are formed by the reaction of ammonia with aqueous chlorine. The use of chloramines was first considered after observing that disinfection by chlorine occurred in two distinct phases. During the initial phase, chlorine-reducing compounds (i.e., demand) caused the rapid disappearance of free available chlorine. However, when ammonia was present bactericidal action was observed to continue, even though the free chlorine residual was dissipated. The subsequent disinfection phase occurs by the action of the inorganic chloramines.

Initially in the early 1900's, chloramines were used for taste and odour control, however, it was soon recognized that chloramines were longer lasting and more stable, but a less powerful disinfectant than free chlorine. Because of this, chloramines were found to be effective for controlling bacterial re-growth in the distribution system. Ammonia shortage during World War II caused the popularity of chloramination to decline, however, concern during the past two decades over chlorinated organics (e.g., THM and HAA formation) in water treatment and distribution systems, increased interest in chloramines because they form very few disinfection by-products (DBPs).

4.6.1. Advantages and Disadvantages

The following list highlights selected advantages and disadvantages of using chloramines as a disinfection method for drinking water. Because of the wide variation of system size, water quality, and dosages applied, some of these advantages and disadvantages may not apply to a particular system.

Advantages

1. Chloramines are not as reactive with organics as free chlorine in forming DBPs. Use of chloramines may reduce total THM concentrations reaching consumers because chloramines do not form THMs on contact with natural organic matter in the water, although it may form other by-products;

2. The monochloramine residual is more stable and longer lasting than free chlorine or chlorine dioxide, thereby providing better protection against bacterial re-growth in systems with large storage tanks and dead end water mains;

3. Chloramines do not tend to react with organic compounds, therefore many systems will experience fewer incidences of taste and odour complaints when using chloramines;

4. Chloramines are inexpensive; and

5. Chloramines are easy to make.

Disadvantages

1. The disinfecting properties of chloramines are not as strong as other disinfectants, such as chlorine, ozone, and chlorine dioxide, and should be considered as a secondary disinfectant only;

2. Use of chloramines may provide less protection from contamination of the distribution system through cross-connections, watermain breaks and other causes;

3. Chloramines cannot oxidize iron, manganese, and sulphides;

4. When using chloramines as the secondary disinfectant, it may be necessary to periodically convert to free chlorine for biofilm control in the water distribution system;

5. Excess ammonia in the distribution system may lead to nitrification problems, especially in dead ends and other locations with low disinfectant residual;

6. Monochloramines are less effective as disinfectants at high pH rather than at low pH;

7. Dichloramines have treatment and operation problems; and

8. Chloramines must be made on-site, but unlike most substances added to water for treatment purposes, chloramines cannot be prepared at high concentrations. It can only be made by the addition of ammonia to lightly pre-chlorinated water, or chlorine to water containing low concentrations of ammonia. Contact between high concentrations of chlorine and ammonia or ammonium salts must be avoided because the sensitive and violently explosive substance, nitrogen trichloride, may be formed.

4.6.2. Converting Treatment Plants to Chloramines

Municipalities that wish to modify disinfectant practices by using chloramines must show the DOEC clear evidence that bacteriological and chemical protection of consumers will not be compromised in any way and that aspects of chloramination mentioned below are considered in any permit application.

Planning

Project planning and preparation are essential to ensure an efficient changeover, to maintain a dependable and safe system, and to preserve the public confidence in the water purveyor. Planning and preparation should consider the following aspects:

1. Raw water composition and suitability to chloramination;
2. Treatment plant and distribution system attributes and monitoring program;
3. Employee training;
4. Public notification and education; and
5. Environmental affects from chloraminated water.

Preliminary Analysis
A bench scale study is necessary to identify the water characteristics and to determine if chloramination is suitable. Some of the study objectives and variables to consider are as follows:
Organic nitrogen in the water;
Ammonia residual desired in the distribution system; and
Chloramine residual type and concentration required in the distribution system.

4.6.3. Potential Operational Impacts of Chloramination

4.6.3.1. Pre-treatment

Ammonia in excess of the required chlorine can promote the growth of nitrifying bacteria in filter beds (i.e., rapid sand filters). The excess ammonia acts as a nutrient and causes the growth of nitrifying bacteria, which convert the excess ammonia to nitrates and nitrites. Excessive levels of nitrate in drinking water may cause serious illness and sometimes death in infants less than six months of age.

4.6.3.2. Nitrification

Nitrification in chloraminated drinking waters is usually partial. Partial nitrification occurs when the chloraminated water has excess ammonia present in the distribution system. Partial nitrification can have various adverse effects on water quality, including a loss of total chlorine and ammonia residuals and an increase in heterotrophic plate count (HPC) bacteria concentration.

4.6.3.3. Taste and Odour

If the chlorine to ammonia-nitrogen ratios are between 3:1 and 5.5:1, disagreeable tastes and odours should be evaluated at the consumer tap. Fishy tastes and odours (e.g., from source waters and return washwater from the washwater treatment system) can be controlled by a 1-hour contact time with free-chlorine residual of 2 mg/L prior to the addition of ammonia. This pre-chlorination eliminates the fishy taste and odour but may increase the THM concentrations at the plant effluent.

4.6.4. Special Considerations for Chloramination Facilities

4.6.4.1. Organic Nitrogen

Concentrations of organic nitrogen and ammonia nitrogen as low as 0.3 mg/L may interfere with the chloramination process. The monochloramine residuals will hydrolyze with the organic nitrogen to form organochloramines, which are non-germicidal. This reaction would take about 30 to 40 minutes. After the monochloramine residuals disappear, free ammonia nitrogen reappears. Free ammonia nitrogen is a powerful biological nutrient. Its presence promotes biological instability in that portion of the distribution system. Biological instability usually results in foul tastes and odours plus dirty and/or coloured water at the consumers tap.

The free chlorine residual or chloramine residual method may be used to clean an area with biological instability. Of the two methods, the free chlorine residuals method is superior. Free chlorine residuals restore distribution system stability quicker (i.e., a few days for free chlorine versus weeks for chloramines), the clean-up process can be monitored, and the clean up is complete when the free chlorine residual concentration reaches 85 % of the total chlorine concentration.

4.6.4.2. Mixing

Thorough and reasonably rapid mixing of chlorine and ammonia in the main plant stream shall be arranged so as to avoid formation of odorous dichloramine. Also, mixing at the point of application can greatly affect the bactericidal efficiency of the chloramine process. When the pH of the water is between 7 and 8.5, the reaction time between ammonia and chlorine is practically instantaneous. If chlorine is mixed slowly into the ammoniated water, organic matter, especially organic matter prone to bleaching with chlorine solution, may react with the chlorine and interfere with chloramine formation.

4.6.4.3. Blending Waters

When chlorinated water is blended with chloraminated water, the chloramine residual will decrease after the excess ammonia has been combined and monochloramine is converted to dichloramine and nitrogen trichloride. The entire residual can be depleted. Therefore, it is important to know how much chlorinated water can be blended with a particular chloraminated water stream without significantly affecting the monochloramine residual. Blended residual curves should be developed for each specific blend.

4.6.4.4. Corrosion

Chloramination and corrosion control can limit bacterial biofilm development in the distribution system. If optimum corrosion of iron pipes is not controlled, the chloramination efficiency may be impacted. Corrosion inhibitors with higher phosphate concentrations may reduce corrosion rates.

4.6.4.5. Formation of Nitrogen Trichloride

If water in the distribution system tends to form nitrogen trichloride, the finished water should be subjected to post-aeration, which readily removes nitrogen trichloride. Nitrogen trichloride is also readily destroyed by sunlight.

4.6.4.6. Human Health and the Environment

Users of kidney dialysis equipment are the most critical group that can be impacted by chloramine use. Chloramines can cause methemoglobinemia and adversely affect the health of kidney dialysis patients if chloramines are not removed from the dialysate water. Chloramines can also be deadly to fish as chloramines in water are considerably more toxic to fish and other aquatic organisms than free chlorine. The residuals can damage the gill tissues, enter the red blood cells, and cause an acute blood disorder. Chloramine residuals should be removed from the water prior to the water contacting any fish. As such, fish hobbyists should be notified, along with pet stores and aquarium supply establishments.

4.6.5. Ammonia Feed Facilities

Ammonia feed facilities can be located on-site at the water treatment plant or at remote locations in the distribution system. Most ammonia feed facilities use either gaseous (anhydrous ammonia) or liquid (aqueous) ammonia. Though anhydrous ammonia is a gas at ambient temperature and pressure, it is commonly stored and transported as a liquid in pressure vessels. In this phase, ammonia is highly soluble in water. Storage facilities and handling equipment should be kept dry

4.6.5.1. Anhydrous Ammonia

Anhydrous ammonia is stored in portable cylinders or stationary tanks. Portable cylinders are similar to chlorine cylinders and are available in 45, 68, and 364 kg sizes. The cylinders are rated for a minimum service pressure of 3300 kPa (480 psi). Stationary tanks are typically 3800 L vessels that can be used on-site and are refilled by tanker trailers (not used in the province).

Anhydrous ammonia is applied using an ammoniator. An ammoniator is a self-contained modular unit with a pressure-reducing valve, gas flow meter, feed rate control valve, and miscellaneous piping for controlling the flow of ammonia. Automatic paced ammoniators are available.

4.6.5.2. Aqueous Ammonia

Aqueous ammonia is not used in the province so it is not discussed in this document. Please contact the DOEC for requirements.

4.6.5.3. Piping and Valving

For anhydrous ammonia, the typical piping materials for both direct and solution feed systems are stainless steel, PVC, and black iron. Stainless steel or black iron pipe is used in the high-pressure (i.e., greater than 103 kPa (15 psi)) portions of the feed system. PVC pipe is used only in the low-pressure portion of the feed system, after the ammoniators.

4.6.6. Safety Provisions for Chloramine Generation Facilities

A chloramination facility should include safety provisions to prevent the formation of nitrogen trichloride and the vaporization of ammonia at ambient temperatures. The possible formation of nitrogen trichloride at a chloramination facility should be considered when selecting sites for the ammonia and chlorine storage facilities.

Chlorine gas and ammonia gas should never be stored in the same room. The ammonia gas application points should be located at least 1.5 m away from chlorine feed solution lines. Anhydrous ammonia is lighter than air, so any leaking vapour will rise quickly. Under pressure, anhydrous ammonia is a liquid. Great amounts of heat are absorbed when the pressurized liquid reverts to a gas. If the storage tanks and/or chemical feed equipment are installed indoors, ventilation and vapour detection devices should be located at high points in the room. Ammonia gas storage tanks should be protected from direct sunlight or direct sources of heat to avoid pressure increases in the tank. Otherwise, ammonia gas may be released into the atmosphere through the pressure relief valves.

4.6.7. Points of Application

The formation of monochloramine can be accomplished by first adding ammonia and then chlorine, or vice versa. Ammonia is added first where the formation of objectionable taste and odour compounds caused by the reaction of chlorine and organic matter is a concern. However, most drinking water systems add chlorine first in the treatment plant in order to achieve the required concentration and contact time (CT) to meet the disinfection requirements. Typically, the point of ammonia addition is selected to “quench” the free chlorine residual after a target period of time based on optimizing disinfection versus minimizing DBP formation. Because the germicidal effectiveness of monochloramine is a factor of 200 less than for free chlorine, extremely long contact times are required for monochloramine to meet disinfection requirements. Therefore, if ammonia is added first, a means of ensuring that CT requirements are met must be developed.

4.6.8. Impact on Other Treatment Processes

Monochloramine addition impacts other processes at the water treatment facility. These impacts include:

1. Ammonia used in the chloramination process can provide nutrient ammonia for nitrifying bacteria growth in the distribution system, which can cause increased nitrate levels in the finished water where systems do not normally test for nitrate;

2. Imbalances in chlorine and ammonia concentrations (in greater than an 8 to 1 ratio) can cause breakpoint chlorination reactions to occur when encountered in distribution system; and

3. Monochloramine addition upstream of filters will reduce biological growth on filters. This has a favourable impact on the filters by keeping them clean and reducing the backwash frequency. It also has the undesirable impact of reducing BDOC removal in the filters when the filters are run in a biological mode.

4.6.9. Environmental Effects

Temperature, pH, and organic and inorganic compounds all play a part in the effectiveness of chloramines. Below is a summary of the affect these parameters have on pathogen inactivation.

Temperature

Similar to most other disinfectants, the bactericidal and viral inactivation efficiency of chloramine increases with increasing temperature. Moreover, the efficiency dramatically decreases under conditions of high pH and low temperature. For example, the inactivation of *E.coli.* is approximately 60 times slower at a pH of 9.5 and temperatures of 2 and 6 °C than at a pH of 7 and temperatures between 20 and 25°C.

pH

The addition of ammonia gas or ammonia solution will increase the pH of the water. The effect of pH on disinfection has more to do with the organism than with the disinfectant, however, pH also impacts disinfection efficiency by controlling the chloramine species distribution. However, pH may be a compounding factor because changes in pH may alter the physiological response of the organism.

The actual pH shift may be small in well-buffered water but the effects on disinfectant power and corrosiveness of the water may require consideration. Ammonia gas forms alkaline solutions, which may cause local plugging by lime deposition. Where hard water is to be treated, a side stream of pre-softened water may be needed for ammonia dilution so as to reduce plugging problems.

Organic Nitrogen and Other Compounds

In addition to ammonia, free chlorine reacts with organic nitrogen compounds to form a variety of organic chloramines. These organic chloramines are undesirable by-products because they exhibit little or no microbiocidal activity. Several other reactions may occur which divert chlorine from the formation of chloramines. These reactions can include oxidation of iron, manganese, and other inorganics such as hydrogen sulphide.

4.6.10. DBP Formation

The effectiveness of chloramines to control DBP production depends upon a variety of factors, notably the chlorine to ammonia ratio, the point of addition of ammonia relative to that of chlorine, the extent of mixing, and pH.

Monochloramine (NH_2Cl) does not produce DBPs to any significant degree, although some dichloroacetic acid can be formed from monochloramine and cyanogen chloride formation is greater than with free chlorine. The inability to mix chlorine and ammonia instantaneously allows the free chlorine to react before the complete formation of chloramines. In addition, monochloramine slowly hydrolyzes to free chlorine in aqueous solution. Therefore, halogenation reactions occur even when monochloramine is formed prior to addition in the treatment process. The closer the chlorine to ammonia ratio is to the breakpoint, the greater the formation of DBPs. In addition to controlling the formation of DBPs, chloramination results in lower concentrations of a number of the other specific organic halides generated from free chlorine, except for cyanogens chloride. The application of chloramines results in the formation of chlorinated organic material, although it occurs to a much lesser degree than from an equivalent dose of free chlorine.

Section 5 – Sewerage Works

5. Sewerage Works

The Engineer shall follow these guidelines and comply with all requirements of the DOEC in the design of any sewerage works project. In addition, the Engineer should confer with the DOEC prior to undertaking the design of any major infrastructure project.

A Technical Report prepared and signed by the Engineer representing the Owner must be submitted to the DOEC. The Technical Report is to be completed in accordance with Section 2 of this document.

5.1. Regulations

Reference should be made to *The Environmental Control Water & Sewage Regulations, 2003* regarding the allowable discharges to a sanitary sewer and receiving environment.

5.2. Sewers and Appurtenances

Sanitary sewers shall be designed of suitable materials, be of sufficient capacity, and be installed at required grades and alignment to carry sewage from residential, commercial, institutional and industrial establishments to the treatment plant or point of disposal. Infiltration of groundwater shall be excluded as far as possible and connection of stormwater sources such as roof, yard, street and foundation drains is prohibited.

5.2.1. Types of Sewers

Sewers shall be classified broadly with respect to use as follows:

Sanitary sewers carry domestic sewage from houses, business, buildings and other public and private establishments. They may be designed to remove certain industrial wastes, but clean cooling waters should not be discharged to sanitary sewers.

Storm sewers carry storm water and surface drainage, street wash and other wash waters or drainage, but exclude sanitary sewage and industrial wastes and effluent from septic tanks or other treatment processes.

Combined sewers receive both sanitary sewage and storm runoff. To the greatest extent possible, these shall be prohibited.

Collector (main) sewers are those sewers to which one or more branch sewers are tributary and which serve as inlets to sewerage works system.

Intercepting sewers receive dry weather flow from a number of transverse sewers or outlets and frequently additional predetermined quantities of storm water (if from a combined system) and carry such water to a point for treatment or disposal.

Outfall sewers receive sewage from a collecting system and/or a sewerage treatment plant and carry it to a point of final discharge.

Criteria for the design of all the above-mentioned sewers, except storm and outfall sewers, are generally the same. Certain features, which are not common to the design of all systems, have been included in this section. Additional design criteria for outfall sewers are discussed in Section 5.2.15.

5.2.2. Capacity of Sewers

The flow rates of sewage or wastewater for which the sewer capacity should be provided shall be determined from careful consideration of the present and future quantities of domestic sewage, commercial and industrial wastes, groundwater infiltration and any other unavoidable contributions. The design of extensions to existing sanitary sewer systems shall be based on experience if adequate records are not available, or when new systems are being established, the design criteria shall be substantiated by data for similar systems. In determining the required capacities of sanitary sewers, the following factors should be considered:

1. Peak hourly rates of flow;

2. Peak rates of flow from industrial plants; and

3. Groundwater infiltration.

5.2.3. Sewage Flows

Sewage 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.

5.2.3.1. Extraneous Sewage Flows

5.2.3.1.1. Inflow

When designing sanitary sewer 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. 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 sewers. Studies have shown that flows from this source can result in gross overloading of sewers, pumping stations and sewage treatment plants for extended periods of time. The DOEC recommends that foundation drainage be directed either to the surface of the ground or into a storm sewer system, if one exists.

5.2.3.1.2. Infiltration

The amount of groundwater leakage directly into the sewer system (infiltration) will vary with the 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 completely eliminated and an allowance must be made in the design sewage flows to cover these flow contributors. Despite the fact that these allowances are generally 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 sewer system. The infiltration allowances used for sewer design should not be confused with leakage limits used for acceptance testing following construction. The latter allowances are significantly lower and apply to a sewer system when the system is new and generally without the private property portions of the building sewers constructed.

5.2.3.2. Design Criteria

5.2.3.2.1. Development with Separate Storm and Sanitary Sewer Systems

Developments using approved piping materials with separate storm and sanitary sewer systems meeting infiltration allowances specified by Department of Municipal and Provincial Affairs *Municipal Water, Sewer and Road Specifications* shall apply a peaking factor to the average daily flow only. There shall be no need to allow for extraneous flows.

5.2.3.2.2. Development with Combined Sewer Systems – Estimating Wastewater Flows

The following sections outline methodologies for quantifying wastewater flows. From both quantitative and a qualitative point of view, owners of wastewater systems are encouraged to develop and implement policies and programs to promote "at source reduction" for any and all contaminants in wastewater.

5.2.3.2.2.1. Residential (Population-Generated)

Every effort should be made to establish design flows using measured data (see Section 5.2.2). As a second preferred option, measured flow data from similar applications may be used. If no flow data exists for the system or data for similar systems exist, the peak (population-generated) flow for a residential population may be determined by the following formula:

$$Q_{pdw} = \frac{G \times P \times Pf}{86.4}$$

Where: Q_{pdw} = peak dry weather design flow rate (L/s);
G = per capita average daily design flow (L/d), for residential use 340 L/capita/day;
P = the design contributing population in thousands; and
Pf = a "peaking factor".

The peaking factor (Pf) should be the larger of 2.5 or Harmon's Peaking Factor where:

$$Harmon's\,Peaking\,Factor = 1 + \frac{14}{4 + P^{0.5}}$$

Where: P = the design contributing population in thousands.

5.2.3.2.2.2. Commercial/Institutional and Industrial

For detailed system design, the average wastewater flows from commercial/ institutional and industrial land use areas are to be estimated as set out in Table 5.1 or by actual documented usage.

Table 5.1
Estimated Sewage Flows

Type of Establishment		Unit	L/day
Residential	Private Dwelling	Person	340
	Apartment Buildings	Person	340
Transient Dwelling Units	Hotels	Bedroom	340
	Lodging Houses and Tourist Homes	Bedroom	270
	Motels and Tourist Cabins	Bedroom	270
Camps	Trailer Camps (Private Bath)	Person	340
	Trailer Camp (Central Bath, etc)	Person	230
	Trailer Camp (Central Bath, Laundry)	Person	300
	Luxury Camps (Private Bath)	Person	340
	Children’s Camps (Central Bath, etc)	Person	230
	Labour Camps	Person	180
	Day Camps-No Meals	Person	70
Restaurants	Average Type (2 × Fire Commissioner’s capacity)	Patron	70
(Incl.Washrooms)	Bar/Cocktail Lounge (2 × Fire Commissioner’s capacity)	Patron	25
	Short order or Drive-In Service	Patron	25
Clubhouses	Residential Type	Person	340
	Non-Residential (Serving Meals)	Person	160
Institutions	Hospitals	Bed	900
	Other Institutions	Bed	375
Schools	Elementary (No Shower or Cafeteria)	Person	50
	With Cafeteria	Person	70
	With Cafeteria and Showers	Person	90
	With Cafeteria, Showers and Laboratories	Person	115
	Boarding	Person	340
Theatres	Theatre (Indoor)	Seat	25
	Theatre (Drive-In With Food Stand)	Car	25
Automobile Service Stations	No Car Washing	Car Served	23
	Car Washing	Car washed	340
Miscellaneous	Stores, Shopping Centres & Office Buildings	M2	6
	Factories (8-hour shift)	Person	115
	Self-service Laundries	Wash	230
	Bowling Alleys	Alleys	900
	Swimming Pools and Beaches	Person	70
	Picnic Parks (With Flush Toilets)	Person	50
	Fairgrounds (based upon average attendance)	Person	25
	Assembly Halls	Seat	25
	Airports (Based on Passenger Use)	Passenger	15
	Churches	Seat	15
	Beauty Parlours	Seat	200
	Barber Shops	Seat	75
	Hockey Rinks	Seat	15

5.2.3.2.2.3. Peak Factor

When using the above estimated sewage flows, 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 will be at an average rate of say 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.

5.2.3.2.2.4. Industrial Sewage Flows

Industrial sewage flows will be decided in consultation with the Pollution Prevention Division, of the DOEC.

5.2.3.2.2.5. Flow Variation

Peak sewage flow rates 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.

5.2.3.2.2.6. Flow Rate

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

5.2.3.2.2.7. Average Flow Generation Estimates for Planning

For system planning purposes, when specific land uses and zoning are unknown and the requirements of Section 5.2.3.2.2.2 cannot be defined, the recommended lower limits for estimation of average flow generation (to be used for preliminary planning unless the use of other values is justified with more specific or reliable information) are as follows:

1. Commercial and Institutional Land Uses - The lower limit for Average Flow Generation should be 40 m^3/day/ha (0.46 L/s/ha);

2. Industrial Land Uses - The lower limit for average flow generation should be 30 m^3/day/ha (0.35 L/s/ha);

3. Determination of Peak Dry Weather Flow Rate - Peak dry weather flow rates for specific design areas are to be determined by application of a peaking factor (Pf), related to the average flow rate (Q_{AVG} in L/s) in accordance with the following expression to a maximum value of 5.0:

$$Pf = 6.659\left(Q_{AVG} - 0.168\right)$$

   Following from this, the peak dry weather flow rate (Q_{PDW} in L/s) may be determined as follows:

$$Q_{PDW} = Pf \times Q_{AVG} = 6.659\left(Q_{AVG} \times 0.832\right)$$

4. Special Considerations - High-Water-Consumption Land Uses - The foregoing guidelines may not be applied to high water consumption land uses such as heavy industry, meat packing plants, breweries, etc. Detailed analysis of the design requirements specific to each development proposal is required in such cases; and

5. Residential Components of Commercial Developments - Where proposed commercial developments include discretionary residential components, the sanitary flow generation from the residential component should be determined in accordance with Section 5.2.3.2.2.1, and is to be included in the determination of the total generation for the development.

5.2.3.2.2.8. Extraneous Flow Allowance - All Land Uses

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

1. General Inflow/Infiltration Allowance - A general allowance of 0.28 L/s/ha 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 next section.

2. Inflow Allowance - 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 flow rate should be increased by 0.4 L/s for each such manhole, which is applicable for manholes which have been waterproofed. For new construction, all sanitary manholes in sag locations are to be waterproofed. 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. Others - In areas where weeping tiles are connected to the sanitary sewer system, an additional amount, based on on-site measurements, should be included in the design flow. The designer should also take into account the pipe material and soil type in determining the extraneous flow allowance.

5.2.3.2.2.9. Total Peak Design Flow Rates

The total peak design flow rates should be the sum of the peak dry weather flow rates as generated by population and land use, and the rate of all extraneous flow allowances, as determined for the design contributing area.

5.2.4. Sewer Size

In no instance shall any street sewer be less than 200 mm diameter.

5.2.5. Sewer Grade

5.2.5.1. Hydraulic Design

It is recommended that sanitary sewers be designed using Manning's formula, with a roughness coefficient *n* of no lower than 0.013 for all smooth-walled pipe materials. Use of lower *n* values may be permitted if deemed justifiable on the basis of research or field data presented. The Manning formula, which is the most commonly used formula for calculating sewer capacity, is as follows:

$$Q = \frac{7.8546 \times 10^{-6}}{n} D^2 R^{2/3} S^{1/2}$$

Where: Q = Flow Capacity of sewer (L/s);
D = Inside diameter of pipe (mm);
R = Hydraulic radius of pipe (mm);
S = Sewer slope; and
n = Roughness factor.

5.2.5.2. Minimum and Maximum Velocities

All sewers shall 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 a “n” value of 0.013. Use of other practical “n” values may be permitted by the DOEC if deemed justifiable. Velocities above 4.5 m/s may be permitted with high velocity protection.

Table 5.2 details the minimum slopes, which will provide a velocity of 0.6 m/s when sewers are flowing full:

Table 5.2
Minimum Sewer Slopes

Minimum Slopes for Full-Pipe Velocity of 0.6 m/s	
Sewer Size (mm)	**Minimum Slope per 100 metres**
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

It is recommended that the actual pipe slopes should not be less than 0.5%.

To achieve 0.6 m/s flow velocities in sewers, which will flow less than ⅓ 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.

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 0.3 of the diameter or greater for design average flow. Whenever such decreased slopes are selected, the design engineer must include with their report, the computations of the anticipated flow velocities of average and daily or weekly peak flow rates. The pipe diameter and slopes shall be selected to obtain the greatest practical velocities to minimize settling problems. The operator of the sewer system will give written assurance to the DOEC that any additional sewer maintenance required by reduced slopes will be provided.

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

5.2.5.3. Sewers on Steep Slopes

Sewers on 20% slopes or greater shall be secured with concrete anchors, or equivalent, spaced as follows:

1. Grades 20% and up to 35% - not over 11 m centre to centre;

2. Grades 35% and up to 50% - not over 7.5 m centre to centre; and
3. Grades 50% and over - not over 5.0 m centre to centre.

5.2.5.4. Sewers in Tidal Zones

Sewers located within tidal zones and under the influence of rising and falling tides must be fitted with a sewage pumping station at the outfall. This is necessary to ensure proper functioning of the sewer system, including adequate flushing, reduction in freezing potential, and problems associated with tidal surge.

5.2.6. Sewer Location

5.2.6.1. Cross-Connection Prohibited

There shall be no physical connections between a public or private potable water supply system and a sewer, or appurtenances thereto which would permit the passage of any wastewater or polluted water into the potable supply. No water pipe shall pass through or come into contact with any part of a sewer manhole.

5.2.6.2. Relation to Water Works Structures

Sewers located in the vicinity of wells or other public water supply or structures should be constructed to conform to regulations governing public water supplies regarding minimum separation distances.

All existing waterworks units, such as basins, wells or other treatment units, within 60 m of the proposed sewer shall be shown on the engineering plans.

Soil conditions in the vicinity of the proposed sewer within 60 m of water supply sources shall be determined and shown on the engineering plans.

5.2.6.3. Relation to Watermains

5.2.6.3.1. Horizontal and Vertical Separation

Refer to Section 3.7.6.

5.2.6.3.2. Crossings

Refer to Section 3.7.6.2.

5.2.6.4. Stream Crossings

5.2.6.4.1. Cover Depth

The top of all sewers entering or crossing streams shall be at sufficient depth below the natural bottom of the streambed to protect the sewer lines. In general, the following cover requirements must be met:

1. 300 mm of cover where the sewer is located in rock;

2. 900 mm of cover in other material. In major streams, more than 900 mm of cover may be required; and

3. In paved stream channels, the top of the sewer line should be placed below the bottom of the channel pavement.

5.2.6.4.2. Horizontal Location
Sewers located along streams shall be located outside of the streambed and sufficiently removed from there to provide for future possible stream widening and to prevent pollution by siltation during construction.

5.2.6.4.3. Structures
The sewer outfalls, headwalls, manholes, gate boxes, or other structures shall be located so they do not interfere with the free discharge of flood flows of the stream.

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

5.2.6.4.5. Materials
Sewers entering or crossing streams shall be constructed of ductile iron pipe with mechanical joints, or similar construction to ensure watertight joints free from change in alignment or grade. Material used to backfill the trench shall be stone, coarse aggregate, washed gravel, or other materials which will not readily erode, cause siltation, damage pipe during placement, or corrode the pipe.

5.2.6.4.6. Siltation and Erosion
Construction methods that will minimize siltation and erosion shall be employed. The design engineer shall include in the project specifications the method(s) to be employed in the construction of sewers in or near streams. Such methods shall provide adequate control of siltation and erosion by limiting unnecessary excavation, disturbing or uprooting trees and vegetation, dumping of soil debris, or pumping silt-laden water into the stream. Specifications shall require that cleanup, grading, seeding and planting or restoration of all work areas shall begin immediately. Exposed areas shall not remain unprotected for more than seven days.

5.2.7. Sewer Alignment
Sewers 600 mm diameter or less shall be laid with straight alignment and uniform grade between manholes. The alignment shall be checked by either using a laser beam or lamping. Where street layouts are such that straight alignment between manholes is impractical, sewers may be "curved" to conform to street curvature by the deflection of straight pipe in accordance with the manufacturer's recommendations. Curved sewers shall be limited to simple curves, which start and end at manholes. The minimum radius of curvature shall be 30 m. An alignment test such as "balling" must be conducted on curved sewers. When curved sewers are proposed, minimum slopes indicated in Section 5.2.5.2 must be increased accordingly to provide a recommended minimum velocity of 0.6 m/s when flowing full. Provision shall be made to provide additional maintenance for curved sewers.

5.2.8. Depth of Sewers

All sewers shall be laid at depths sufficient to drain basements and to be protected against damage by frost and traffic. Where, for specific reasons, shallow depths are necessary and can be justified, the sewer shall be protected to prevent damage by frost or traffic and insulated to prevent freezing. Sewers laid in deep or excessively wide trenches shall be adequately reinforced to prevent damage.

It is recommended that sewers be laid at a depth of at least 2.1 m. A minimum depth of 1.5 m may be permitted under some circumstances. Local soils and weather must be taken into account at all times.

5.2.9. Sewer Material

Any material generally accepted for sewers will be given consideration, but the material selected shall be suitable for the local conditions, such as the chemical characteristics of the sewage, character of industrial wastes, possibility of septicity, soil characteristics, exceptionally heavy external loadings, abrasion, corrosion, the necessity for reducing the number of joints, soft foundations, and other similar problems.

Suitable couplings complying with ASTM specifications shall be used for jointing dissimilar materials. The leakage limitations on these joints shall be in accordance with Section 5.2.11.

All sewers shall be designed to prevent damage from super-imposed live, dead and frost induced loads. Proper allowances for loads on the sewer shall be made because of soil and potential groundwater conditions, as well as the width and depth of trench.

Where necessary, special bedding, haunching and initial backfill, concrete cradle, or other special construction shall be used to withstand anticipated potential superimposed loading or loss of trench wall stability. See ASTM D2321 or ASTM C 12 when appropriate.

For new pipe materials for which ASTM standards have not been established, the design engineer shall provide complete pipe specifications and installation specifications developed on the basis of criteria adequately documented and certified in writing by the pipe manufacturer to be satisfactory for the specific detailed plans.

5.2.9.1. Warning/marker and Detection Tape

Warning/marker and detection tape as specified in the Department of Municipal and Provincial Affairs Water, Sewer and Roads Master Specification Section 02223.2.1 and detailed drawings numbered 0290 and 0300, shall be installed continuously with a minimum 1.0 m overlap at joints above water, sewer, and forcemains. Warning/marker tape shall be heavy gauge polyethylene, 150 mm wide and indicate the service line below. Detectable tape shall 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 300 mm below ground surface and be detectable using conventional pipe location apparatus.

5.2.10. Overflow Structures and Bypasses

Backflow-preventing devices shall be provided whenever flooding of the sewer outlet may be possible.

The locations of bypasses shall be shown clearly on the plans and adequate details presented in order that the operation of such bypasses may be evaluated.

5.2.11. Sewer Testing and Inspection

Each section of a sanitary sewer shall 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, shall be flushed prior to testing. The following tests are recommended as applicable.

5.2.11.1. Exfiltration Test

The exfiltration test shall be conducted as follows:

1. Fill test section with water in such a manner as to allow displacement of air in the line;

2. Immediately prior to test period add water to pipeline until there is a head of 1 m over the interior crown of the pipe measured at the highest of the test section or water in the manhole is 1500 mm above static ground water level, whichever is greater;

3. Duration of exfiltration test shall be 1 hour; and

4. Water loss at end of test period shall not exceed maximum allowable exfiltration over any section of pipe between manholes.

5.2.11.2. Infiltration Test

Infiltration tests shall 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.

The infiltration test shall be conducted as follows:

1. Install a watertight plug at upstream end of pipeline test section;

2. Discontinue pumping operations for at least 3 days before test measurements are to commence and during this time keep thoroughly wet at least one third of pipe invert perimeter;

3. Prevent damage to pipe and bedding material due to flotation and erosion;

4. Place a 90° V-notch weir, or other measuring device approved by the DOEC in invert of sewer at each manhole; and

5. Measure rate of flow over a minimum of 1 hour, with recorded flows for each 5 minute interval

5.2.11.3. Allowable Leakage

Allowable leakage shall be determined by the following formula:

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

where: L = allowable leakage (L/hr);
D = diameter (mm);
S = Length of section (m); and
F = leakage factor (L/hr/mm of diameter/100 m of sewer).

Exfiltration Test:

Porous Pipe F = 0.12 L
Non-Porous Pipe F = 0.02 L

Infiltration Test:

Porous Pipe F = 0.10 L
Non-Porous Pipe F = 0.02 L

5.2.11.4. Low Pressure Air Testing

Air testing equipment shall 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 shall be filled with air until a constant pressure of 28 kPa is reached. After a 2-minute period the air supply shall be shut off, and the pressure decreased to 4 kPa. The time required for the pressure to reach 17 kPa shall be measured.

5.2.11.5. Allowable Time for Air Pressure Decrease

Minimum times allowed for air pressure drop are provided in Table 5.3:

Table 5.3
Minimum Times Allowed for Pressure Drop

Pipe Diameter (mm)	Minimum Time (min:sec)
100	1:53
150	2:50
200	3:47
250	4:43
300	5:40
375	7:05
450	8:30
525	9:55
600	11:20

5.2.11.6. Sewer Inspection

The specifications shall include a requirement for inspection of manholes and sewers for watertightness, prior to placing into service.

Video inspections shall be conducted in accordance with the *Municipal Water, Sewer and Roads Specifications*.

5.2.12. Sanitary Sewer Service Connections

Sewer services shall be consistent with the local or provincial plumbing and drainage regulations. It is required that, unless tees or Y's have been installed, saddles be used in connecting the service pipe to the sewer. Generally, these are placed at an angle of 45° above the horizontal. If a saddle type connection is used, it shall be a device designed to join with the types of pipe, which are to be connected. All materials used to make service connections shall be compatible with each other and with the pipe materials to be joined and shall be corrosion proof. Connections shall be made by qualified personnel only.

5.2.13. Manholes

Sewer manholes are mainly for the purpose of facilitating maintenance and operation of the sewer system and shall be designed, constructed and located as follows:

5.2.13.1. Minimum Diameter

Minimum diameter for manholes shall be 1200 mm; larger diameters are preferred for larger sewers.

5.2.13.2. Manhole Covers

Minimum cover clear opening shall be 580 mm. Watertight covers should be used where manholes will be subject to flooding. Where significant sections of sewers are provided with watertight manholes, extended vents may be required to prevent excessive sulphide generation. Locked manhole covers may be desirable in isolated easement locations or where vandalism may be a problem.

5.2.13.3. Location

Manholes shall be located at all junctions, changes in grade, size or alignment (except with curvilinear sewers) and termination points of sewers.

The maximum acceptable spacing for manholes is 90 to 120 m for sewers 200 to 450 mm in diameter. Spacings of up to 150 m may be used for sewers 450 mm to 750 mm in diameter. Larger sewers may use greater manhole spacing.

Cleanouts may be used only with approval from DOEC and shall not be substituted for manholes nor installed at the end of laterals greater than 45 m in length.

5.2.13.4. Drop Manholes

Drop manholes should be provided for lateral sewers entering a manhole at an elevation of 600 mm or more above the manhole invert. Where the difference between the incoming sewer and the manhole invert is less than 600 mm, the invert should be filleted to prevent solid deposition.

Drop manholes shall be constructed with an outside or inside drop connection. Inside drop connections shall be secured to the interior wall of the manhole and provide access for cleaning as per the *Municipal Water, Sewer and Roads Specifications*.

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

5.2.13.5. Channel and Benching

The flow channel through manholes should be made to conform in shape and slope to that of the sewers. The channel walls should be formed or shaped to the full height of the crown of the outlet sewer in such a manner to not obstruct maintenance, inspection or flow in the sewers.
When curved flow channels are specified in manholes, including branch inlets, minimum slopes indicated in Section 5.2.5.2 should be increased to maintain acceptable velocities.

A bench shall be located on each side of any manhole channel when the pipe diameter(s) are less than the manhole diameter. The bench should be sloped no less than 4%. The direct connection of sewer service lines to manholes is prohibited unless the service enters at the flow line of the manhole.

5.2.13.6. Manhole Steps

Manhole rung spacing shall be not less than 300 mm or more than 375 mm. Their minimum width should be 375 mm and they should be designed to prevent the foot from slipping sideways. Adequate clearance should be provided between the wall of the manhole and the rungs to afford secure footing.

5.2.13.7. Watertightness

Manholes shall be of the pre-cast concrete or poured-in-place concrete type. Manholes shall be waterproofed on the exterior. Manhole lift holes and grade adjustment rings shall be sealed with non-shrinking mortar or other acceptable material.

Inlet and outlet pipes shall be joined to the manhole with a gasketed, flexible, watertight connection or any watertight connection arrangement that allows differential settlement of the pipe and manhole wall to take place.

Watertight manhole covers are to be used wherever the manhole tops may be flooded by street runoff or high water. Locked manhole covers may be desirable in isolated easement locations or where vandalism may be a problem.

5.2.13.8. Frost Lugs

Where required, frost lugs should be provided to hold pre-cast manhole sections together.

5.2.13.9. Safety Chains

Safety chains should be provided on the downstream side of manholes for sewers 1200 mm in diameter or greater.

5.2.13.10. Inverted Siphons

The use of inverted siphons shall be kept to a minimum, but where they must be used, they shall consist of at least two lines, one of which shall be not less than 200 mm in diameter. Inverted siphons should be designed in size and grade to maintain a velocity of at least l m/s under conditions of average dry weather flow. Under minimum dry weather flow, the independent operation of one of the lines shall provide a minimum velocity of l m/s. Where the above conditions cannot be met, other means shall be provided. They shall be provided with necessary appurtenances for maintenance, convenient flushing and cleaning equipment. The inlet and discharge structures shall have adequate clearances for cleaning equipment, inspection and flushing. The inlet and outlet details shall be arranged so that the normal flow is diverted to one barrel and so that either barrel may be cut out of service for cleaning. The vertical alignment should permit cleaning and maintenance.

5.2.14. Alternative Wastewater Collection Systems

The use of alternative wastewater collections systems will be considered only when the conventional wastewater system is either impractical or cost prohibitive. The consultant will have to convince the DOEC regarding the viability and long-term sustainability of the proposed alternative wastewater system.

5.2.14.1. Applications

Under a certain set of circumstances, each alternative system has individual characteristics, which may dictate standards for usage. Each potential application should be analyzed to determine which system is most cost effective and which will comply with local requirements. The following features of various sewerage alternatives are considered in a planning a project.

5.2.14.2. Population Density

Conventional sewers are typically costly on a lineal meter basis. When housing is sparse, resulting in long reaches between services, the cost of providing conventional sewers is often prohibitive. Pressure sewers, small diameter gravity sewers and vacuum sewers are typically less costly on a lineal meter basis, and often prove to be more cost-effective when serving sparse populations.

5.2.14.3. Ground Slopes

Where the ground profile over the main slopes continuously downward in the direction of flow, conventional or small diameter gravity sewers are normally preferred. If intermittent rises in the profile occur, conventional sewers may become cost prohibitive. The variable grade gravity sewer variation of small diameter gravity sewers, by use of inflective gradients and in conjunction with *septic tank effluent pump* (STEP) 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. *Small Diameter Gravity Sewers* (SDGS) are buried less deep, owing to the flatter gradients permitted. Pressure sewers or vacuum sewers are often found to be practical in flat areas, as ground slope is of little concern. In areas where the treatment facility or interceptor sewer are higher than the service population, pressure sewers and vacuum sewers are generally preferred, but should be evaluated against SDGS systems with lift stations.

5.2.14.4. Subsurface Obstacles

Where rock excavation is encountered, the shallow burial depth of alternative sewer mains reduces the amount of rock to be excavated. Conventional sewers require deep excavation, and may sometime encounter groundwater. Depending on the situation, dewatering can be expensive and difficult to accomplish.

5.2.14.5. Low Pressure Collection Systems

In certain areas primarily due to the very rocky terrain servicing using conventional gravity sewers has been found to be economically unacceptable. An alternative to the use of gravity sewers is the use of a pressure sewer system.

5.2.14.5.1. Definition

A pressure sewer system uses small diameter, when compared with a conventional gravity sewer, polyethylene pressure pipe to convey sewage to a central treatment or collection point. Each property discharges into a common pressure pipe using a specially designed grinder pump, usually installed in the basement or crawl space of the house.

5.2.14.5.2. Design

5.2.14.5.2.1. Grinder Pump

The design of a pressure sewer system is totally dependent upon the efficiency and reliability of the grinder pump. The pump has to be designed for this specific application and must be capable of the following:

1. Grinding the waste to a fine slurry to enable pumping through small diameter mains. The pump must be capable of routinely handling items which are commonly found in domestic sewage, for example, plastic, wood, rubber and light metal objects;

2. As each pump will be located at a different point along the common pressure main, at various elevations and might operate either individually or in unison with several other pumps, it is essential that it be able to operate consistently over a very wide range of heads which are continually and often rapidly changing; and

3. Capability of operating at least 25% above the low-pressure sewer system design criteria of 280 kPa. This is based upon the maximum daily number of pumps operating simultaneously. The simultaneous operation of more than the design maximum number of pumps is usually a transitory occurrence, however, it is essential that no damage occur to the pumping equipment, pipelines or appurtenances.

5.2.14.5.2.2. Storage Tank

The storage tank is to be sized to provide a pump "on-off" cycle most desirable for the efficient and durable operation of the pump. The sizing should minimize the potential for the generation of septic sewage due to long retention periods.

5.2.14.5.2.3. Collection Pipe

As required by the Department of Municipal and Provincial Affairs, *Municipal Water, Sewer and Roads Specifications*.

5.2.14.5.2.4. Frost Protection

The majority of pressure sewer systems are constructed in shallow trenches, to minimize the quantity of rock excavation. Adequate protection must be provided to prevent freezing using a combination of insulation and heat tracing. The method of pipe installation must be of proven design for the specific application.

5.2.14.5.2.5. Common Trench Construction
The requirements of Section 5.3.20.6 must be adhered to.

5.2.14.5.3. Applications for Approval
All applications for the approval of a pressure sewer system should contain, as well as the information described in Section 1.0, the following additional information:

1. A detailed cost analysis of the pressure sewer system alternative as compared to conventional servicing, including:
   a) Capital cost;
   b) Purchase and installation cost of the individual pump units; and
   c) Operation and maintenance costs.

2. The municipality in which the system is to be installed must agree with the proposal to utilize pressure sanitary sewers;

3. The municipality must indicate that it has either:
   a) Adequately trained personnel on staff for the purpose of repair and maintenance of the individual pumping units and the collection system; or
   b) Has executed an operative/maintenance agreement with a local company, which has trained personnel in this respect.

4. Approval from each homeowner to permit access to the pump unit for servicing, when it is contained within the house. (Note: It is possible to house the pump unit within a chamber external to the house, however, this adds to the capital cost of the system); and

5. Complete details of the type of pump to be used including complete performance data and proof of reliability for this specific application. The pump supplier must maintain an adequate supply of spares and replacement units in the Province of Newfoundland and Labrador.

5.2.14.6. Vacuum Sewer Systems
Vacuum sewer systems consist of a vacuum station, collection piping, wastewater holding tanks, and valve pits. In these systems, wastewater from an individual building flows by gravity to the location of the vacuum ejector valve. The valve seals the line leading to the main in order to maintain required vacuum levels. When a given amount of wastewater accumulates behind the valve, the valve opens and then closes allowing a liquid plug to enter the line. Vacuum pumps in a central location maintain the vacuum in the system.

5.2.14.6.1. Services
Each home on the system should have its own holding tank and vacuum ejector valve. Holding tank volume is usually 115 L. As the wastewater level rises in the sump, air is compressed in a sensor tube, which is connected to the valve controller. At a preset point, the sensor signals for the vacuum valve to open. The valve stays open for an adjustable period of time and then closes. During the open cycle, the holding tank contents are evacuated. The timing cycle is field

adjusted between 3 and 30 seconds. This time is usually set to hold the valve open for a total time equal to twice the time required to admit the wastewater. In this manner, air at atmospheric pressure is allowed to enter the system behind the wastewater. The time setting is dependent on the valve location since the vacuum available will vary throughout the system, thereby governing the rate of wastewater flow.

The valve pit is typically located along a property line and may be combined with the holding tank. These pits are usually made of fibreglass, although modified concrete manhole sections have been used. An anti-flotation collar may be required in some cases.

5.2.14.6.2. Collection Piping

The vacuum collection piping usually consists of 100 mm and 150 mm mains. Smaller 75 mm mains are not recommended as the cost savings of 75 mm versus 100 mm mains are considered to be insignificant.

Rubber gasketed PVC pipe, which has been certified by the manufacturer as being suitable for vacuum service is recommended. Solvent welding should be avoided when possible. The mains are generally laid to the same slope as the ground with a minimum slope of 0.2 percent. For uphill transport, lifts are placed to minimize excavation depth. There are no manholes in the system; however, access can be gained at each valve pit or at the end of a line where an access pit may be installed. Installation of the pipe and fittings follows water distribution system practices. Division valves are installed on branches and periodically on the mains to allow for isolation when troubleshooting or when making repairs. Plug valve and resilient wedge gate valves have been used.

5.2.14.6.3. Vacuum Station

Vacuum stations are typically two-story concrete and block buildings approximately 7.5 m x 9 m in floor plan. Equipment in the station includes a collection tank, a vacuum reservoir tank, vacuum pumps, wastewater pumps, and pump controls. In addition, an emergency generator is standard equipment, whether it is located within the station, outside the station in an enclosure, or is of the portable, truck mounted variety.

The collection tank is made of either steel or fibreglass. The vacuum reservoir tank is connected directly to the collection tank to prevent droplet carryover and to reduce the frequency of vacuum pump starts. Vacuum pumps can be either liquid ring or sliding vane type and are sized for a 3 - 5 hour/day run-time. The wastewater discharge pumps are non-clog pumps with sufficient net positive suction head to overcome tank vacuum. Level control probes are installed in the collection tank to regulate the wastewater pumps. A fault monitoring system alerts the system operator should a low vacuum or high wastewater level condition occur.

5.2.14.7. Small Diameter Gravity Sewers

Small diameter gravity sewers (SDGS) require preliminary treatment through the use of an interceptor or 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 are laid with variable or inflective gradients. Fewer manholes are used and most are replaced with cleanouts except at major junctions to limit infiltration/inflow and entry of grit. The required size and shape of the mains is dictated primarily by hydraulics rather than solids carrying capabilities.

5.2.14.7.1. House Connections

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

5.2.14.7.2. Interceptor Tanks

Interceptor tanks are buried, watertight tanks with baffled inlets and outlets. They are designed to remove both floating and settleable solids from the waste stream through quiescent settling over a period of 12-24 hours. 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 as an interceptor tank.

5.2.14.7.3. Service Laterals

Service laterals connect the interceptor tank with the collector main. Typically, they are 75-100 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.

5.2.14.7.4. Collector Mains

Collector mains are small diameter plastic pipes with typical minimum diameters of 75-100 mm. The mains are trenched into the ground at a depth sufficient to collect the settled wastewater from most connections by gravity. Unlike conventional gravity sewers, small diameter gravity sewers 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 gradeline. Also, the alignment may be curvilinear between manholes and cleanouts to avoid obstacles in the path of sewers.

5.2.14.7.5. Cleanouts, Manholes, and Vents

Cleanouts, manholes, and vents provide access to the collector mains for inspection and maintenance. In most circumstances, 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.

5.2.14.7.6. Lift Stations

Lift stations are necessary where the elevation differences do not permit gravity flow. Either STEP units (see Section 5.2.14.3) or mainline lift stations may be used. STEP 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.

5.2.15. Outfall Sewers

5.2.15.1. Approval

The location and discharge from an outfall sewer could have a major impact on the environment. The selection of the various parameters to be used for the design of an outfall would be included in the Technical Report (Section 2).

Site-specific studies may be required where:

1. A municipal system collects and processes industrial waste;
2. Effluent discharges to a fresh water body;
3. Municipal waste or effluent discharges to sensitive marine environment; and
4. Any other situation as deemed appropriate by the DOEC.

5.2.15.2. Design

The objective of an outfall is to introduce the effluent stream into the receiving water in a place and manner chosen to achieve efficient mixing with that receiving water. Outfalls must be located, designed, constructed and maintained for efficient mixing of discharges with receiving waters. The design must be based on the analysis of salinity, temperature and dissolved oxygen profiles, wind speed and direction, tidal currents, and other relevant oceanographic measurements, and must take into consideration both positive and negative buoyant plumes. An outfall shall not impact on any intake for fish plants, shellfish beds, aquaculture areas, recreational areas, and any other sensitive areas.

5.2.15.2.1 Length and Depth

Outfall lengths, depths, and depth-distance combinations for marine discharges of municipal wastewater, are given in Table 5.4. A length-depth combination less than that specified in Table 5.4 may be permitted where the proposed treatment and site-specific studies can demonstrate that acceptable dilutions can be attained to meet the receiving water quality guidelines.

5.2.15.2.2. First Point of Discharge

The distance to the first point of discharge from the mean low water mark must be equal to or greater than 30 m.

5.2.15.2.3. Protection and Maintenance

The outfall sewer shall be designed and constructed to protect against the effects of floodwater, tides, ice or other hazards as to reasonably insure its structural stability and freedom from stoppage. A manhole should be provided at the shore end of all gravity sewers extending into the receiving waters. Hazards due to navigation must be considered in designing outfall sewers.

Table 5.4
Depth and Length Requirements for Marine Outfalls

Discharge Rate (Q) (Litres/day)		Depth and Length for Outfalls (m)		
		Shellfish, Intakes and Aquaculture	*Recreational*	*Other*
Q # 50,000	Depth Length	Study Required	5 50	3 30
50,000 < Q # 350,000	Depth Length	Study Required	6 75	5 50
>350,000	Depth Length	Study Required	Study Required	Study Required

Length refers to the distance from the low water mark to the point of discharge or the first diffuser nozzle.
* Depth refers to the distance between the low normal spring tide level to the top of the outfall.

5.2.15.2.4. Signage

Signs are to be posted to indicate the location of outfalls within 100 m.

5.2.15.2.5. Offset from Bottom

An appropriate offset is to be maintained between the discharge point and bottom of the receiving water body, to ensure that plume is not affected by the bed of the water body and vice versa.

5.2.15.2.6. Sampling Provisions

All outfalls shall be designed so that a sample of the effluent can be obtained at a point after the final treatment process and before discharge to or mixing with the receiving water.

5.2.15.3. Receiving Water Quality Objectives

The typical level of treatment required for any new treatment plant in the province is secondary treatment with disinfection. However, each new plant will be evaluated on a case-by-case basis. Required levels of treatment may be determined to be higher or lower than secondary treatment based on waste assimilation studies. The procedure for carrying out these studies is described in the following section.

5.2.15.3.1. Assimilation Study Procedures

1. Level of Effort - As part of the pre-design evaluation, the engineer shall determine in consultation with DOEC the level of effort required for the particular wastewater assimilation study. The department may conclude that the effects of the proposed project on the receiving water will be minimal. In this case, the department will set effluent limitations based upon a simple model (possibly basic dilution calculations). In this case, only a minimal level of effort is required for the receiving water study (RWS). The department will determine which parameters will require measurement.

When the department is unsure of the possible effects of a project on the receiving water, it may require that an intermediate RWS and model simulation be conducted as a preliminary assessment tool. If the results of this study indicate that the proposed project would have only a minor effect on the receiving water, the department may, at that point, set effluent limitations. The department shall set the data requirements for the intermediate RWS's.

The third level of effort that may be required is a detailed RWS and complex modelling application. The results of this procedure will determine required effluent limitations. This approach will be required when the department believes that a proposed project may have a significant impact on the receiving water quality. The department will determine RWS data requirements.

2. Water Sampling Procedures - Instruments for electronic *in-situ* determination of water quality parameters should be calibrated at least before and after each sampling trip. For example, samples should be collected for salinity to verify field measurements and samples fixed in the field for dissolved oxygen to verify dissolved oxygen probes.

   All field collection equipment should be listed and prepared before each sampling trip, insuring that all collection containers are clean and proper log forms and labelling equipment are available. Different containers should be available for metals, nutrients, organics, dissolved oxygen, etc. due to their cleaning and preservation requirements.

   An established sequence of collection should be developed and maintained throughout the monitoring effort, insuring that new personnel are trained in the proper methods and sequence of data collection. All samples should be logged and sample log sheets should include station location, time, depth, results of *in situ* sampling, and container numbers for each type of sample. Datum should always be clearly specified (e.g. time of day standard, datum for water surface elevations).

   All samples should be preserved on board, where the preservation technique will vary with the type of analysis required, but may involve icing, acidification, organic extraction, etc. The preservation techniques should be documented prior to implementation to the monitoring study. For some samples that do not preserve well it may be necessary to either conduct analyses on board or quickly transfer them to nearby on-shore facilities.

   Additional samples should be collected to determine sampling variability and individual samples may be split prior to analysis to determine analytical variability. The number of replicate samples should be established as part of the planning for the monitoring effort. Field samples may also be spiked with a known amount of a standard prior to analysis. The identity of the spiked, split and duplicate samples should be kept on separate logs and the analyst should not be aware of their identity.

   The samples should be transferred from the field to the laboratory in a timely manner. The field logs should be recorded and a laboratory log kept of the samples and their arrival. Custody sheets may be kept to further document the transferral of samples.

5.2.15.3.2. Assimilation Capacity of Receiving Water

In essence, the dilution/assimilative capacity of a waterbody depends on wastewater characteristics and a host of physical, chemical and biological factors, such as the flow or volume of the waterbody and the wastewater discharges, dispersion of effluent, depth and width of the waterbody, type of substrate, algal growths, benthic deposits or organic sludges, etc.

A wastewater assimilation study is the mechanism to be used in estimating a waterbody's assimilative capacity and establishing effluent requirements to meet the Canadian Environmental Quality Guidelines (CEQG). Either simple dilution formulae or more sophisticated mathematical models can be used as assessment techniques, depending on the circumstances. For example, with a dilution ratio greater than 20 to 1, simple dilution formulae may be adequate for estimating effluent requirements for discharges with a high degree of treatment (e.g. secondary treatment) and which do not contain hazardous substances. With a dilution ratio less than 20 to 1, more complex assessment techniques may be required to estimate assimilative capacity. Further, under complex situations (e.g. multiple uses of water, flood control requirements, etc.) sophisticated mathematical models may be used to estimate assimilative capacity and effluent requirements.

In areas with existing water quality better than CEQG, it is a good general principle not to allocate the entire assimilative capacity of a receiving waterbody. The need for maintenance of a reserve capacity should be established on a case-by-case basis.

In addition to meeting the CEQG, a thorough receiving water assessment may be required before the discharge of effluent containing toxic substances will be permitted. Such an assessment should include studies of the potential accumulation and concentration of the substances in the environment (such as bed sediments and aquatic flora and fauna), synergistic effects with other substances and physical factors (such as temperature changes or radiant energy) that may affect the environmental impact of contaminants.

5.2.15.3.2.1. Dilution Ratio

Dilution ratio is a simple measure of a receiving water's assimilative capacity. Dilution ratios should be based upon the 7 consecutive day average low streamflow occurring once in 20 years (7Q20), and the peak hourly effluent discharge rate (both expressed in the same units).

5.2.15.3.2.2. Mixing Zone

A mixing zone is a region of a waterbody in which an effluent discharge with quality (chemical/physical/biological) characteristics different from those of the receiving water is in transit and is progressively assimilated from the immediate outfall area to the outer limits of the region. At the boundaries or outer limits of the mixing zone, water quality objectives established by the DOEC to protect beneficial water uses should be achieved. A mixing zone may not be used as an alternative to adequate treatment. Existing biological, chemical, physical and hydrological conditions should be known when considering the location of a new mixing zone or limitations on an existing one.

No conditions within the mixing zone should be permitted which:

1. Are rapidly lethal to important aquatic life (resulting in conditions which result in sudden fish kills and mortality of organisms passing through the mixing zones);
2. Cause irreversible responses which could result in detrimental postexposure effects;
3. Result in bioconcentration of toxic materials which are harmful to the organism or its consumer; or
4. Attract organisms to the mixing zones, resulting in a prolonged and lethal exposure period.

The mixing zone should be designed to satisfy the following conditions:
1. Shall allow an adequate zone of passage for the movement or drift of all stages of aquatic life (specific portions of a cross-section of flow or volume may be arbitrarily allocated for this purpose);
2. Shall not interfere with the migratory routes, natural movements, survival, reproduction (spawning and nursery areas), growth, or increase the vulnerability to predation, of any representative aquatic species, or endangered species;
3. Eliminate rapid changes in the water quality, which could kill organisms by shock effects;
4. Total loading from all mixing zones within a waterbody must not exceed the acceptable loadings from all point source discharges required to maintain satisfactory water quality;
5. Mixing zones should not result in contamination of natural sediments so as to cause or contribute to exceedances of the water quality objectives outside the mixing zone

The mixing zone shall be:
1. Free from substances in concentrations or combinations which may be harmful to human, animal or aquatic life;
2. Free from substances that will settle to form putrescent or otherwise objectionable sludge deposits, or that will adversely affect aquatic life or waterfowl;
3. Free from debris, oil, grease, scum or other materials in amounts sufficient to be noticeable in the receiving water;
4. Located so as not to interfere with fish spawning and nursery areas;
5. Free from colour, turbidity or odour-producing materials that would:
   a) Adversely affect aquatic life or waterfowl;
   b) Significantly alter the natural colour of the receiving water;
   c) Directly or through interaction among themselves or with chemicals used in water treatment, result in undesirable taste or odour in treated water, and;
   d) Free from nutrients in concentrations that create nuisance growths of aquatic weeds or algae or that results in an unacceptable degree of eutrophication of the receiving water;

5.2.15.3.2.2.1. Calculating the Mixing Zone

The mixing zone should be as small as practicable, and shall not be of such size or shape to cause or contribute to the impairment of existing or likely water uses. Mixing zone size shall be established on a case-by-case basis, but in no case shall it exceed the following:

For marine bodies of water the following, measured from the point of discharge and from the mean low water mark, apply for the purpose of calculating the mixing zone:

1. The depth/height is the distance from the bed to the surface of the water; and

2. The radius is, either:
   a) 100 m; or
   b) 25% of the width of the body of water, whichever is less.

For an estuary the following, measured from the point of discharge and from the mean low water mark, apply for the purpose of calculating the mixing zone:

1. The depth/height is the distance from the bed to the surface of the body of water;

2. The width, perpendicular to the path of the stream, is the lesser of:
   a) 100 m; or
   b) 25% of the width of the stream or estuary; and

3. The length, parallel to the path of the stream, is the distance between a point 25 metres upstream and a point, which is the lesser of:
   a) 175 m downstream; or
   b) A distance downstream at which the width of the effluent plume equals the width determined above for an estuary (Section 5.2.15.3.2.2.1 (2)).

5.2.15.3.3. Waste Assimilation Study Field Procedures for Coastal Waters

5.2.15.3.3.1. General Strategies

1. Before any fieldwork is carried out on any coastal water, the problem(s) should first be defined, and the objective(s), of the study laid out.

2. Maps and hydrographic charts of the proposed discharge area shall be obtained, as well as any previous reports on the waterbody and any nearby municipal and/or industrial discharges.

3. All existing data on water quality monitoring, water uses, current speed and direction, receiving water density distribution, and volumes and characteristics of waste discharges should be obtained.

5.2.15.3.3.2. Pre-Design Surveys vs. Monitoring Surveys

Oceanographic surveys for wastewater disposal systems can be placed in one of two general categories - predesign or monitoring. Each of the two types of surveys has a different objective and possesses unique requirements that demand careful consideration.

Predesign surveys must provide not only the necessary information to determine the proper alignment of the submarine outfall, the location and orientation of the wastewater diffuser system, but also the final design criteria for the outfall and diffuser system which will assure the protection and enhancement of the receiving environment. In addition, accurate bathometric profiles, benthic soil characteristics for outfall placement and foundation, and sediment erosion and deposition behaviour must be determined.

Monitoring surveys are of several types. Predischarge monitoring surveys are conducted to establish the baseline or natural conditions of the receiving water, receiving sediments and adjacent shoreline prior to discharge of wastewaters. Post discharge monitoring surveys are conducted for the purpose of determining the effects of the wastewater discharge on the receiving environment or for the purpose of assessing the performance of the disposal system for verification of design criteria and future design improvements.

5.2.15.3.3.3. Pre-Design Waste Assimilation Studies

5.2.15.3.3.3.1. General Objectives

Three major objectives must be satisfied in the conduct of a pre-design oceanographic survey. Firstly, a pre-design survey must determine the dispersion or diluting characteristics of the receiving water. Secondly, a pre-design survey must provide sufficient information on the ecosystem in the proposed discharge area to assure that biologically significant or sensitive areas will not be adversely affected by the disposal system, both during construction of the outfall and during the continuing discharge of the wastewater. Thirdly, foundation conditions for outfall and diffuser placement must be determined prior to preparation of engineering plans and specifications on the waste disposal system.

To satisfy all three objectives a general area for placement of the disposal system must be surveyed and potential alternative outfall sites chosen.

Because the rational determination of an outfall length necessary to meet particular receiving environment requirements for a specified level of treatment will be dependent upon the survey results, the pre-design oceanographic survey should be designed to provide the greatest practical flexibility in the selection of outfall alignments and lengths.

5.2.15.3.3.3.2. Measurement of Parameters

Those parameters, which are most critical to the design, should be most thoroughly measured in the conduct of the oceanographic survey, with less effort expended in assessing more traditional parameters, which have a much lesser effect on design considerations. The critical parameters include the following:

1. Density of Receiving Water - Density measurements throughout the water column are required in order to estimate the extent of initial dilution occurring over a diffuser. Water density may be determined by calculation from temperature and salinity, conductivity or specific measurements.

2. Horizontal Ocean Currents - Horizontal current velocity may be measured by using propeller or cone-type meters suspended at a specific depth.

3. Horizontal Eddy Dispersion - Measurement and prediction of the magnitude of horizontal eddy dispersion requires determination of horizontal ocean currents occurring in the horizontally moving wastewater field and of the appropriate eddy diffusivity. Measurements of the diffusivity, or diffusion coefficient, are very difficult and require sophisticated and complex techniques. The relative dilution effected by eddy dispersion of concern to most wastewater disposal systems, however, is small, permitting a rather gross estimate of the diffusivity without affecting the design substantially.

   The effect of wave climate on surface and alongshore currents should also be evaluated. The effect of ice accumulation around outfall pipes should also be analyzed.

4. Decay/Disappearance Rates - Determination of disappearance rates for specific non-conservative constituents requires, in most cases, special studies wherein a mass of the discharged wastewater containing the constituent is monitored in the receiving environment over a period of time to determine its decay as a function of time. The observed diminution must be corrected for physical dilution that has occurred by eddy dispersion over the period of observation. A tracer material, usually non-toxic fluorescent dye, should be employed in these studies to obtain physical dilution.

5. Wind Velocity and Direction

6. Receiving Water Quality Parameters - To assure placement of an outfall in an area which will provide the least deleterious effects on the environment and to predict what effects the wastewater discharge will have on the receiving environment, it is necessary to characterize the physical and chemical conditions of the receiving environment and the indigenous flora and fauna in the inter-tidal zones, in the benthic sediments and in the overlying waters.

   Water quality characterization should include dissolved oxygen, pH, temperature, salinity, transparency or turbidity, total and fecal coliforms, BOD, and in some cases, nitrogenous and phosphorous forms. Benthic sediments should be characterized with respect to particle size distribution, organic carbon and nitrogen, dissolved sulphide, heavy metals, and chlorinated hydrocarbons.

   Microplankton, macroplankton and nekton populations in the receiving waters may be sampled and their diversity determined. The benthos should be properly examined for specific enumeration, and the extent of biological productivity and diversity should be determined.

7. Benthic Soil - Prior to preparation of engineering plans and specifications the structural characteristics of the benthic soils must be determined. For ease and economics of construction of the outfall, the alignment should not encounter rock outcroppings, and other submerged obstructions, abrupt vertical discontinuities or escarpments. Coring and soil analysis should be performed to determine footing characteristics, and bathometric profiles

should be made over a period of time to determine if problems with shoaling and shifting sediments are likely to occur.

5.2.15.3.3.3.3. Survey Procedure

Because oceanographic surveys for pollution control facilities are usually restricted by economic and time constraints, it is extremely important to specify a survey program that will provide the most useful and meaningful data within the resources available, and should include the following:

1. Sampling Stations - Sampling station location should be selected to provide adequate coverage of the receiving environment and should be for the most part located at coordinates representing potential diffuser locations and outfall alignments. Sampling points for certain measurements within the water column should be selected to provide the best representative sample, or samples, of the entire column or of the characteristic under consideration.

2. Current Measurements - Synoptic current measurements throughout the receiving water mass over an extended period of time will provide the best description of the current structure and circulation pattern, but sufficient resources are generally not available for such extensive sampling. A reasonable assessment of a current regime at any particular time can be obtained by taking measurements of current speed and direction near the bottom and top of the water column, and at mid-depth. If the number of current metering points is further limited because of time or other constraints, single-depth current measurements will be most useful at a depth representative of the currents responsible for horizontal movement away from the diffuser following the initial dilution process. In those instances where a pronounced pyenocline exists, the best single depth for measurement would be several meters below the pyenocline. Where there is no such pyeriocline, single depth measurements at several meters below the surface would usually provide the most useful information.

   Coastal currents are affected by lunar tides, major oceanic currents, wind stress, and tributary freshwater discharges, which are all variable in magnitude and effect with respect to time. To obtain a reasonable estimate of the current velocity distribution, therefore, measurements should be taken over a sufficiently long period (preferably one year) to account for the diurnal and seasonal changes.

3. Water Quality Measurements - Water quality characteristics are less variable than current characteristics, but do vary somewhat diurnally and greatly seasonally. Thus, only one or two measurements of water quality characteristics are generally required during a single day several times during a year.

   Water quality characteristics can be measured either *in-situ* with direct reading or recording devices or from discrete collected samples. Existing equipment allows *in-situ* simultaneous measurement of DO, pH, transmittance, temperature and conductivity, and easily provides the necessary vertical definition to establish density gradients and pycnoclines. Other measurements, such as nutrient concentrations, must be made from discrete samples collected with an appropriate water-sampling device.

4. Biological Parameters - Of the biological parameters, the benthic flora and fauna are the least affected by diurnal and seasonal factors, and thus can be characterized adequately by sampling only several times during a year. Benthic samples can be collected either remotely from a vessel or directly by divers. Remote sampling is performed either with a dredge, which allows for recovery of a disturbed sample, or a coring tool, which provides a relatively undisturbed specimen. Generally, dredges are used for biological characterization of the sediments and corers are employed for physical and chemical assay of the benthic materials.

   Several samples are usually taken per time in order to provide an indication of localized sample variation.

   A more systematic and specific approach, but also more biased, to benthos characterization is provided by divers who can observe and report on general biological conditions and obtain samples and specimens for later analysis that are highly controlled with respect to size and location.

   Because the distributions of the most motile and the floating forms, e.g. microplankton, macroplankton and nekton, are highly time and space dependent, a large number of samples for these organisms must be collected to obtain a statistically significant characterization.

   Plankton sampling is usually accomplished by vertical or horizontal tows with appropriately sized netting. Discrete samples collected with conventional water samplers can also be used for plankton enumeration and identification, but this procedure allows for much greater sampling error due to the generally large spatial variations encountered in plankton populations and distributions.

5.2.15.3.3.3.4. Equipment
Current meters can be classed either as moored or non-moored, and can be direct reading or recording. Moored recording meters provide an almost continuous record of current velocity, but are restricted because each individual meter is fixed in both the vertical and horizontal plane. Meters operated from a vessel, on the other hand, provide measurements from a variety of depths and locations, but present only an instantaneous sampling of the current regime.

5.2.15.3.4. Waste Assimilation Study Field Procedures for Estuaries
1. An estuary is defined as the tidal mouth of a large river. Before any fieldwork is carried out on an estuary, the problem(s) should first be defined, and the objectives of the study laid out.

2. Maps and aerial photographs of the survey area should be obtained, as well as any previous reports on the waterbody and municipal and/or industrial discharges. All discharges to the waterbody should be pinpointed on the map. All existing data on water quality monitoring, water takings and consumption, water uses, flows, and volumes and characteristics of waste discharges should be obtained.

3. If manpower and time permit, the entire reach of the estuary to be studied should be inspected. Waste discharges, dispersion patterns of effluents, physical characteristics of the estuary, water uses, algal growths, the presence of benthic deposits or organic sludges and any other pertinent characteristics should be noted.

4. Boundary condition data are external to the model domain and are driving forces for model simulations. For example, atmospheric temperature, solar radiation and wind speeds are not modelled but are specified to the model as boundary conditions and drive modelled processes such as mixing, heat transfer, algal growth, reaeration, photolysis, volatilization, etc. Non-point and point source loadings as well as inflow water volumes are model boundary input. The boundaries at the upstream end of the estuary and the open boundary at the ocean provide major driving forces for change. Models do not make predictions for the boundary conditions but are affected by them.

5. In setting limits on wastewater quantity and quality, the following factors affecting estuarine water quality should be assessed: salinity, sediment, bacteria and viruses dissolved oxygen depletion, nutrient enrichment and over-production, aquatic toxicity, toxic pollutants and bioaccumulation and human exposure:
   a) Salinity is important in determining available habitat for estuarine organisms. Large wastewater discharges into relatively small estuaries or embayments can alter the local salinity regime through dilution. Even when the salinity is not affected by the discharge, it is measured and modelled in order to quantify advection and dispersion. These processes help determine how wastewater is assimilated into the estuary.

   b) Sediment enters estuaries from many sources, and can alter the habitat of benthic organisms. Sediment is also an important carrier of such pollutants as hydrophobic organic chemicals, metals, and nutrients.

      Sediment transport can move pollutants upstream, or between the water column and the underlying bed. Even when wastewater does not introduce excess sediment into an estuary, it is often measured and modelled in order to quantify the transport of sediment-bound pollutants.

   c) Bacteria and viruses may enter estuaries in runoff from farms and feedlots and in effluent from marinas as well as from municipal or industrial wastewater discharges. These pathogens may be transported to bathing beaches and recreational areas, causing direct human exposure and possibly disease. Pathogens also may be transported to shellfish habitat; there they may accumulate in oysters, clams, and mussels and, subsequently, cause disease when eaten by humans.

   d) Adequate, sustained DO concentrations are a requirement for most aquatic organisms. Seasonal or diurnal depletion of DO, then, disrupts or displaces estuarine communities. Ambient DO levels are affected by many natural processes, such as oxidation of organic material, nitrification, diagenesis of benthic sediments, photosynthesis and respiration by phytoplankton and submerged aquatic vegetation, and reaeration. The natural balance can be disrupted by excessive wastewater loads of organic material, ammonia, and nutrients. Other sources of nutrients, such as runoff from agricultural, residential, and urban lands and atmospheric deposition, can also disrupt the DO balance. Excessive heat input from power plants can aggravate existing problems. Because of its intrinsic importance, and because it is affected by so many natural and man influenced processes, DO is perhaps the best conventional indicator of water quality problems.

e) Adequate concentrations of nitrogen and phosphorus are important in maintaining the natural productivity of estuaries. Excessive nutrient loading, however, can stimulate overproduction of some species of phytoplankton, disrupting the natural communities. Periodic phytoplankton "blooms" can cause widely fluctuating DO concentrations, and DO depletion in benthic and downstream areas. Nutrient loads can be introduced in wastewater and runoff and through atmospheric deposition.

f) High concentrations of ammonia, many organic chemicals, and metals can disable or kill aquatic organisms. Acute toxicity is caused by high exposure to pollutants for short periods of time (less than four days). The toxicity of a chemical can be affected by such environmental factors as pH, temperature, and sediment concentrations. Overall toxicity results from the combined exposure to all chemicals in the effluent and the ambient waters.

g) Lower concentrations of organic chemicals and metals that do not cause aquatic toxicity can be taken up and concentrated in the tissues of estuarine organisms. As fish predators consume contaminated prey, bioaccumulation of these chemicals can occur. This food chain contamination can persist long after the original chemical source is eliminated. Humans that regularly consume tainted fish and shellfish can receive harmful doses of the chemical.

   Human exposure to harmful levels of organic chemicals and metals can also occur through drinking water withdrawals from fresh water tidal rivers.

5.2.15.3.5. Waste Load Allocation Modelling
Because of the wide array of variable elements that must be considered in assessing a receiving water's assimilative capacity, computerized mathematical models are generally employed to make the necessary calculations. In the simplest situations, manual calculations can be performed. In most cases, however, the use of computerized mathematical models will be much more convenient.

5.2.15.3.5.1. Model Selection
The initial step of any waste load allocation study is to define the nature and the extent of the problem. Once this is done, the preferred approach in model selection is to use the simplest model that can be applied to a particular case.

Ideally, the model should include only those phenomena that are operative and important in the receiving water being modeled. The most appropriate procedure for selecting a model is to first define the phenomena that are important for the particular site-specific analysis to be performed. Activities that help to define phenomena that should be incorporated include the following:

1. review of existing data on waste loads, and receiving water quality; and
2. preliminary mass balance calculations using simple models or equations that provide analytical solutions for various load sources (combined sewer overflows, non-point sources, sediment) and reaction phenomena.

It is also desirable to attempt to anticipate the technical issues with respect to control actions (level of treatment, alternate discharge locations, etc.) and determine whether this will influence the types of reactions that will be important. From the foregoing, the analyst will generally be able to establish the phenomena that should be included in the selected model and the time and space scale of the analysis, which is most appropriate.

Under ideal circumstances, one would select a formal model or analysis approach that included all the phenomena determined to be important in the study area, and which excluded those reactions that are insignificant in the case in question. While this guidance should be followed as much as possible, in practice a calculation framework or model may be selected because it is available or familiar to the analyst.

In such cases, two criteria are important to apply. First, the model selected must be capable of handling all of the important site-specific phenomena considering the time and space scale of the analysis and using the equations and formulations specified. Secondly, provision should be made, where possible, to eliminate from the calculation framework the effect of any phenomena that are insignificant in the site-specific analysis. In some cases, inclusion of phenomena judged to be unimportant on a site-specific basis can increase the level of uncertainty of the analysis and thus directly affect decisions. In these situations, additional data collection, sensitivity runs, and other aspects of the overall waste load allocation program must be considered, in order that phenomena contained in the calculations are adequately addressed.

Additional evaluation criteria for model selection include completeness of computer program documentation, costs for manpower, and computer time.

5.2.15.3.5.2. Model Selection Guidelines

Guidelines for selection of a model fall under two categories: technical and operational. The technical guidelines ultimately are concerned with matching the model capabilities to the important physical and biochemical processes of the prototypical system. The operational guidelines are concerned with the ease and cost associated with model operation.

The following is the sequence of model selection guidelines (both technical and operational), with a brief discussion of the considerations involved:

1. Technical Guideline #1 - Determine Important Features of the Prototypical System that are Required in the Analysis

   Site-specific data should be collected and reviewed to understand the system and establish the important factors associated with the identified problem. Valuable information can also be obtained from other experienced professionals, especially those who have modelling experience or site-specific field experience, and from personal site visits.

2. Technical Guideline #2 - Review Available Models and Model Capabilities

   There are a wide number of models available capable of performing waste load allocations. It is important to be aware of those capabilities that involve a substantial increase in complexity.

3. Technical Guideline #3 - Match Important Features of the Prototypical System with Model Capabilities - An important step in model selection is comparing the important features of the prototypical system with the model capabilities and selecting, as technically acceptable, those models whose capabilities match the features of the system. A rule of thumb is to select the simplest model(s) that retains all-important features in the prototypical system. Choosing a more complex model is not cost effective since data requirements and computer cost tend to increase rapidly. An overly complex program will not usually result in an improved simulation and may increase uncertainty in the analysis.

4. Technical Guideline #4 - Confirm Selection of Technically Acceptable Models

   To confirm that the models are indeed technically appropriate, the potential user should consult the user's manual and other support documents, contact and discuss the potential application with the DOEC, and consult with other experienced professionals.

5. Operational Guideline #1 - Selection of Candidate Models Based on Ease of Application

   Once a technically acceptable model has been selected, it is necessary to estimate the ease of applying it. However, it is very difficult to evaluate the adequacy of documentation and support and realistically estimate costs without prior experience with the model. Therefore, it is recommended that the DOEC be consulted. It may be possible that special support arrangements (including short courses or informational or personnel exchanges) are available under existing agreements or otherwise could be made available to the potential user. The DOEC may also be able to provide the potential user with a list of local users who could be contacted for information regarding their past or current experience with the computer program associated with the model.

6. Operational Guideline #2 - Selection of Candidate Models Based on Cost of Application and Problem Significance

   It is difficult to estimate overall costs involved in a model application because each application differs in scope and complexity, and the ability to solve or avoid certain problems is very dependent on the experience and technical background of the analysts involved. However, machine requirements and costs associated with typical runs are usually estimated in the program documentation. As a rule, the simpler the model, the less expensive it is to apply. Again, it is essential that the DOEC and other experienced professionals be contacted for information or assistance.

   Once an estimate of the costs of application has been made, it should be compared with the benefits of using the program as part of the water quality modelling effort and the overall importance of the problem. In other words, the WLA study costs should be consistent with the economic, social, or environmental values associated with the problem and its solution.

7. Operational Guideline #3 - Selection of Candidate Models Based on Data Availability and Data Acquisition Costs

   All models require data for input, calibration, and verification. It is best if model selection is not restricted by availability of data and the decision is made to acquire the specific type of data required for the model. On the other hand, if data availability is a constraint, selection of a less sophisticated model than would be warranted on technical grounds may be appropriate.

The first step in model selection is to determine which programs are technically acceptable, based on an understanding of the important physical and biochemical processes in the prototypical system. The second step is to determine the ease and costs of application of those that are technically acceptable. The result of the second step is a list of candidate models that may or may not be ranked according to convenience and cost. The final selection of the preferred model from the list of candidates is based on the overall judgment of the potential user taking into account all of the factors discussed.

5.2.15.3.5.3. Modelling Procedures

The typical development of a site-specific water quality model follows the following three main steps:

1. Initial Assessment - The historical data are reviewed and employed in conjunction with initial model runs, which compare calculated and observed water quality to:
   a) Confirm existing or future water quality problems;
   b) Define the loads, sources, and sinks that control water quality;
   c) Define the important reactions that control water quality; and
   d) Define issues in the area of transport that must be resolved.

   The initial assessment is the first step of the process aimed at understanding the factors controlling water quality. The initial assessment activity is a first full step in understanding quantitatively the factors controlling water quality. It is not a preliminary analysis; instead, the initial understanding is translated into a field and experimental program whose data output begins to challenge and strengthen the understanding of the system.

2. Field Program - This task translates the results of the initial assessment into a practical field program that can be carried out on the receiving water and in the laboratory using the resources and manpower required and/or available.

3. Model Calibration - Following the selection of an appropriate model and the collection of the relevant field data, it is necessary to calibrate the model. Model calibration is necessary because of the semi-empirical nature of present day water quality models. In model calibration activities, the data from the field program are employed to define model criteria, constants and equations. Water quality calculations using the model are developed for the various conditions associated with each of the water quality data sets. These conditions

include those associated with the historical data and the data collected in the field program. Adjustments in the value of model parameters must be made in a consistent fashion for all conditions.

The results of these activities are a set of consistent model parameters, which are then employed to develop water quality calculations for the conditions associated with all available data sets. Comparisons of calculated and observed water quality profiles should be developed. The model runs and calculations employed to search for and define the series of consistent coefficients should be retained since they can provide an indication of system sensitivity.

5.2.15.4. Registration Under the Environmental Assessment Regulations

A trunk sewer pipeline outfall (Sewage/Wastewater Outfall) with 200 mm inside diameter or greater and with a tributary population of 1000 or more or equivalent flow shall be registered.

5.3. Wastewater Pumping Stations

5.3.1. Energy Requirements

In view of the rising costs of energy and the possibility of future energy shortages, designers should attempt to minimize the number of wastewater pumping stations required in sewage collections systems. Deeper gravity sewers, inverted siphons, or aerial sewers may not only eliminate the need for pumping stations, but may prove to be economically more attractive in the long term and should be examined.

5.3.2. Flooding

Wastewater pumping station structures, electrical and mechanical equipment shall be protected from physical damage during major floods and remain fully operational.

5.3.3. Accessibility and Security

The pumping station shall be readily accessible by maintenance vehicles during all weather conditions. The facility should be located off the traffic way of streets and alleys. It is recommended that security fencing and access hatches with locks be provided.

5.3.4. Grit

Where it is necessary to pump wastewater prior to grit removal, the design of the wet well and pump station piping shall receive special consideration to avoid operational problems from the accumulation of grit.

5.3.5. Safety

Adequate provision shall be made to effectively protect maintenance personnel from hazards. Equipment for confined space entry in accordance with Occupational Health and Safety requirements shall be provided for all wastewater pumping stations.

5.3.6. Design

The following sub-sections should be given consideration in the design of wastewater pumping stations.

5.3.6.1. Type

Wastewater pumping stations in general use, fall into four types: wet well/dry well, submersible, suction lift and screw pump, however as pumping and power requirements increase wet well/dry well type stations are preferred. The typical efficiencies of the two types of pumping systems, along with capital, operation and maintenance costs should be considered when choosing between the two types of stations.

5.3.6.2. Access and Safety Landings

Suitable and safe means of access shall be provided to dry wells and to wet wells. Access to wet wells containing either bar screens or mechanical equipment requiring inspection or maintenance shall conform to Section 5.5.1.2. Removable ladders may be provided in small stations where it is impractical to install stairways.

Safety landings shall be installed in all manholes and pumping stations as per the Department of Municipal and Provincial Affairs, *Municipal Water, Sewer and Roads Specifications*.

5.3.6.3. Buoyancy

Where high groundwater conditions are anticipated, buoyancy of the wastewater pumping station structures shall be considered and, if necessary, adequate provisions shall be made for protection.

5.3.6.4. Construction Materials

Materials shall be selected that are appropriate under conditions of exposure to hydrogen sulphide and other corrosive gases, greases, oils and other constituents frequently present in wastewater. This is particularly important in the selection of metals and paints. Contact between dissimilar metals should be avoided or other provisions made to minimize galvanic action.

5.3.6.5. Separation

Dry wells including their superstructure shall be completely separated from the wet well. Common walls must be gas tight.

5.3.6.6. Equipment Removal

Structures and equipment shall be constructed so that there will be convenient facilities to remove pumps and motors. Large pumps shall be accessible to track cranes or suitable portable block and tackle equipment.

5.3.6.7. Dry Space

Dry space should be provided, sufficient in size to accommodate the pumps, motors, valves, piping, etc., without congestion for operation or repairs and maintenance. The floor should slope to a sump equipped for positive drainage.

5.3.7. Pumps

5.3.7.1. Number of Pumps

Each pumping station shall be provided with at least two pumps, and in large main pumping stations, three or more pumps shall be provided.

5.3.7.2. Type of Pumps

The pumps shall be designed specially for pumping sewage and should be set below the high water level of the wet well in order to be self-priming. Otherwise, the pump should be so placed that under normal operating conditions, it will operate under a positive suction pressure.

5.3.7.3. Pump Capacities

If only two units are provided, they shall have the same capacity. Each shall be capable of handling flows in excess of the expected maximum flow. In larger pumping stations where three or more pumps are provided, they should be designed to fit actual flow conditions and shall be of such capacity that, with the largest unit out of service, the remaining units can pump the maximum flow of sewage. Where discharge is directly into the treatment plant, pump capacities should be so selected that one pump will operate almost constantly. The other pumping units shall automatically go into operation whenever the flow exceeds the capacity of the first pump. The pumps should be on a preset cycling arrangement so that one pump is not always the duty pump.

5.3.7.4. Pumping Rates

The pumps and controls of main pumping stations and especially those operated as part of a treatment works, should be selected to operate at varying delivery rates which will permit discharging sewage from the station to the treatment works at approximately its rate of delivery to the pump station. The station design capacity shall be based on peak hourly flow and should be adequate to maintain a minimum velocity of 0.6 m/s in the force main.

5.3.7.5. Pump Connections

The pump suction and discharge lines shall be equipped with suitable check and gate valves to facilitate repairs and shall in no case be less than 100 mm in diameter. Full closing valves shall be installed on the suction and discharge piping to each pump, and a check valve shall be installed in the discharge line of the pump. Where space will not permit the installation of all types of valves, the connecting lines shall be equipped with indicators to show their open and closed positions. Potable water lines shall not be directly connected to any sewage pump, suction or discharge line, for priming, flushing, lubricating, or for any other purpose.

5.3.7.6. Pump Openings

Pumps handling raw wastewater shall be capable of passing spheres of at least 75 mm in diameter.

5.3.7.7. Dry Well Dewatering

A sump pump equipped with dual check valves shall be provided in the dry well to remove leakage or drainage with discharge above the maximum high water level of the wet well. 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 leakage shall be piped or channelled directly to the sump. The sump pump shall be sized to remove the maximum pump seal water discharge, which would occur in the event of a pump seal failure.

5.3.7.8. Electrical Equipment

Electrical systems and components (e.g., motors, lights, cables, conduits, switch boxes, control circuits, etc.) in raw water wastewater wet wells, or in enclosed or partially enclosed spaces where hazardous concentrations of flammable gases and vapours may be present, shall comply with the Canadian Electrical Code requirements. In addition, equipment located in the wet well shall be suitable for use under corrosive conditions. Each flexible cable shall be provided with a watertight seal and separate strain relief. A fused disconnect switch located above ground shall be provided for the main power feed for all pumping stations. When such equipment is exposed to weather, it shall meet the requirements of weatherproof equipment. Lightning and surge protection systems should be considered. A 110-volt power receptacle to facilitate maintenance shall be provided inside the control panel for lift stations that control panels outdoors. Ground fault interruption protection shall be provided for all outdoor outlets.

5.3.7.9. Intake

Each pump shall have an individual intake. Wet well and intake design should be such to avoid turbulence near the intake and to prevent vortex formation.

5.3.7.10. Controls

Control float tubes and bubbler lines should be located, as not to be unduly affected by turbulent flows entering the well or by the turbulent suction of the pumps. Provision shall be made to automatically alternate the pumps in use.

5.3.8. Piping and Valves

5.3.8.1. Piping

Flanged pipe and fittings shall be used for exposed piping inside the pump stations. A flexible connection shall be installed in the piping to each pump so that the pump may be removed easily for repairs. A semi-flexible wall sleeve is recommended where a pipe passes through a wall of the station.

5.3.8.2. Suction Line

Suitable shutoff valves shall be placed on the suction line of dry pit pumps.

5.3.8.3. Discharge Line

Suitable shutoff and check valves shall be placed on the discharge line of each pump. The check valve shall be located between the shutoff valve and the pump. Check valves shall be suitable for the material being handled and shall be placed on the horizontal portion of discharge piping except for ball checks, which may be placed in the vertical run. Valves shall be capable of withstanding normal pressure and water hammer.

All shutoff and check valves shall be operable from the floor level and accessible for maintenance. Outside levers are recommended on swing check valves.

5.3.9. Wet Wells

5.3.9.1. Divided Wells

Where continuity of pump station operation is important, consideration should be given to dividing the wet well into multiple sections, properly interconnected, to facilitate repairs and cleaning.

5.3.9.2. Size

The effective capacity of the wet well below the inlet sewer, where possible, shall provide a holding period of between 10 and 30 minutes for the average flow. To avoid septicity problems wet wells should not provide excessive retention times. Wet wells should be of minimum size consistent with pump capacities and not be designed to provide storage in cases of power failure. Alternatively, the retention period of a wet well may be 5 to 10 minutes when variable speed pumping is employed, and when the number of pumps is adequate.

Wet well sizing will be influenced by factors such as the volume required for pump cycling; dimensional requirements to avoid turbulence problems; the vertical separation between pump control points; the inlet sewer elevation(s); capacity required between alarm levels and basement flooding and/or overflow elevation; number of and horizontal spacing between pumps.

To minimize pumping costs and wet well depth, normal high water level (pump start elevation) may be permitted to be above the invert of the inlet sewer(s) provided basement flooding and/or solids deposition will not occur. Where these problems cannot be avoided, the high water level (pump start elevation) should be approximately 300 mm below the invert of the inlet sewer. Low water level (pump shut down) should be at least 300 mm or twice the pump suction diameter above the centre lien of the pump volute. 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 elbow.

5.3.9.3. Floor Slope

The bottom of the wet well should slope sharply to the suction pipe of the pump to minimize the accumulation of sewage solids. The horizontal area of the hopper bottom shall be no greater than necessary for proper installation and function of the inlet. Slopes of 1.0 vertical to 1.0 horizontal are required.

5.3.9.4. Hazardous Area

Wet wells are classified as Hazardous Location and the requirements of the Canadian Electrical Code must be satisfied for all electrical installations in wet wells, or the equipment must be CSA approved for use in sewage-wet wells.

5.3.9.5. Air Displacement

Covered wet wells shall have provisions for air displacement to the atmosphere, such as an inverted “J” tube or other means.

5.3.10. Safety Ventilation

Ventilation shall be provided for all pumping stations, including both wet and dry wells. Mechanical ventilation shall be provided for all dry wells below the ground surface, and for wet wells where equipment is located. There shall be no interconnection between the wet well and dry well ventilation systems.

5.3.10.1. Air Inlets and Outlets

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

5.3.10.2. Electrical Controls

Switches for operation equipment should be marked and located conveniently. All intermittently operated ventilation equipment shall be interconnected with the respective pit lighting system. Consideration should be given also to automatic controls where intermittent operation is used. The manual lighting/ventilation switch shall override the automatic controls. For a two speed ventilation system with automatic switch over where gas detection equipment is installed, consideration should be given to increasing the ventilation rate automatically in response to the detection of hazardous concentrations of gases or vapours.

5.3.10.3. Fans, Heating and Dehumidification

The fan wheel shall be fabricated from non-sparkling material. Automatic heating and dehumidification equipment shall be provided in all dry wells. The electrical equipment and components shall meet the requirements of Section 5.3.7.8.

5.3.10.4. Wet Wells

Wet well ventilation may be either continuous or intermittent. Ventilation, if continuous shall provide at least 12 complete air changes per hour. Air shall be forced into the wet well by mechanical means rather than solely exhausted from the wet well. The air change requirements shall be based on 100 percent fresh air. Portable ventilation equipment shall be provided for use at submersible pump stations and wet wells with no permanently installed ventilation equipment.

5.3.10.5. Dry Wells

Dry well ventilation may be either continuous or intermittent. Ventilation if continuous, shall provide at least 6 complete air changes per hour; if intermittent, at least 30 complete air changes per hour. A system of two-speed ventilation with an initial ventilation rate of 30 changes per hour for 10 minutes and automatic switch over to 6 changes per hour may be used to conserve heat. The air change requirements shall be based on 100 % fresh air.

5.3.11. Flow Measurement

Suitable devices for measuring flow shall be provided at all pumping stations. Flow control mechanisms shall be located so that flow currents created by the entering sewage or by pump suction will not adversely affect them. Provision shall be made to prevent floating material in the wet well from interfering with the operation of the controls. When a float tube is installed in the dry well, its height shall be sufficient so as to prevent overflow of sewage into the dry well. At larger pumping stations, consideration should be given to installing suitable devices for measuring sewage flows and power consumption. Indicating, totalizing and recording flow measurement shall be provided at pumping stations with a 75 L/s or greater design peak flow. Elapsed time meters used in conjunction with annual pumping rate tests may be acceptable for pump stations with a design peak hourly flow up to 75 L/s provided sufficient metering is configured to measure the duration of individual and simultaneous pump operation. These instruments shall be capable of being integrated (electrically) with automatic samplers.

5.3.12. Potable Water Supplies

There shall 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.

5.3.13. Suction-Lift Pump Stations

Suction-lift pumps shall meet the requirements of Section 5.3.6.

5.3.13.1. Pump Priming and Lift Requirements

Suction-lift pumps shall be of the self-priming or vacuum-priming type. Suction-lift pump stations using dynamic suction lifts exceeding the limits outlined in the following sections may be approved 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 safety factor of at least 1.8 m.

Self-priming pumps shall be capable of rapid priming and re-priming at the “lead pump on” elevation. Such self-priming and re-priming shall be accomplished automatically under design operating conditions. Suction piping should not exceed the size of the pump suction and shall not exceed 7.6 m in total length. Priming lift at the “lead pump on” elevation shall 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 shall not exceed 6.7 m.

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

5.3.13.2. Equipment, Wet Well Access, and Valving Location

The pump equipment compartment shall be above the grade or offset and shall be effectively isolated from the wet well to prevent a hazardous and corrosive sewer atmosphere from entering the equipment compartment. Wet well access shall not be through the equipment compartment and shall be at least 580 mm in diameter. Gasketed replacement plates shall be provided to cover the opening to the wet well for pump units removed for servicing. Valving shall not be located in the wet well.

5.3.14. Submersible Pump Stations

Submersible pump stations shall meet the requirements as outlined in the previous sections, except as modified below.

5.3.14.1. Construction

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

5.3.14.2. Pump Removal

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

5.3.14.3. Electrical Equipment

5.3.14.3.1. Power Supply and Control Circuitry

Electrical supply, control and alarm circuits shall be designed to provide strain relief and to allow disconnection from outside the wet well. Terminals and connectors shall be protected from corrosion by location outside the wet well or through use of watertight seals. If located outside, weatherproof equipment shall be used.

5.3.14.3.2. Controls

The motor control centre shall be located outside the wet well, be readily accessible, and be protected by a conduit seal or other appropriate measures meeting the requirements of the Canadian Electrical Code, to prevent the atmosphere of the wet well from gaining access to the control centre. The seal shall be so located that the motor may be removed and electrically disconnected without disturbing the seal. When such equipment is exposed to weather, it shall meet the requirements of weatherproof equipment.

5.3.14.3.3. Power Cord

Pump motor power cords shall be designed for flexibility and serviceability under conditions of extra hard usage and shall meet the requirements of the Canadian Electrical Code standards for flexible cords in wastewater pump stations. Ground fault interruption protection shall be used to de-energize the circuit in the event of any failure in the electrical integrity of the cable. Power cord terminal fittings shall be corrosion-resistant and constructed in a manner to prevent the entry of moisture into the cable, shall be provided with strain relief appurtenances, and shall be designed to facilitate field connecting.

5.3.14.4. Valves

Check valves that are integral to the pump need not be located in a separate valve pit provided that the valve can be removed from the wet well in accordance with Section 5.3.14.2.

5.3.15. Alarm Systems

Alarm systems shall be provided for pumping stations. The alarm shall be activated in cases of power failure, sump pump failure, pump failure, unauthorized entry, or any cause of pump station malfunction. Pumping station alarms shall be telemetered to a municipal facility that is manned 24 hours a day. If such a facility is not available and a 24-hour holding capacity is not provided, the alarm shall be telemetered to municipal offices during normal working hours and to the home of the responsible person(s) in charge of the lift 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.

5.3.16. Emergency Operation

Pumping stations and collection systems shall be designed to prevent or minimize bypassing of raw sewage. For use during periods of extensive power outages, mandatory power reductions, or uncontrolled storm events, consideration should be given to providing a controlled, high-level wet well overflow to supplement alarm systems and emergency power generation in order to prevent backup of sewage into basements, or other discharges which may cause severe adverse impacts on public interests, including public health and property damage. Where a high level overflow is utilized, consideration shall also be given to the installation of storage/detention tanks, or basins, which shall be made to drain to the station wet well. Storage capacity should be related to frequency and length of power outages for the area. Power outage history can be obtained from the power supply company for the grid where the pump station is to be located. This data can then be utilized with peak design flows for design of storage facilities to minimize overflow. Where elimination of overflows is not practical this data can be used to predict frequency and quantities of overflows. Where public water supplies, shellfish production, or waters used for culinary or food-processing purposes exist, overflows shall not be permitted.

5.3.16.1. Overflow Prevention Methods

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

1. 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); and

2. An in-place or portable pump, driven by an integral combustion engine meeting the requirements of Section 5.3.17, capable of pumping from the wet well to the discharge side of the station.

5.3.16.2. Overflows

Overflows are not permitted unless special written permission has been obtained from the DOEC.

5.3.17. Equipment Requirements

5.3.17.1. General

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

5.3.17.1.1. 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 shall be capable of shutting down the engine and activating an alarm on-site and as provided in Section 5.3.15. Protective equipment shall monitor for conditions of low oil pressure and overheating, except that oil pressure monitoring will not be required for engines with splash lubrication.

5.3.17.1.2. Size

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

5.3.17.1.3. Fuel Type

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

5.3.17.1.4. Engine Ventilation

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

5.3.17.1.5. Routine Start-up

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

5.3.17.1.6. Protection of Equipment
Emergency equipment shall be protected from damage at the restoration of regular electrical power.

5.3.17.2. Engine-Driven Pumping Equipment
Where permanently installed or portable engine-driven pumps are used, the following requirements in addition to general requirements shall apply.

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

5.3.17.2.2. Operation
The engine and pump shall be equipped to provide automatic start-up and operation of pumping equipment. Provisions shall 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 5.3.17.2.3.

5.3.17.2.3. 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 shall be provided. A riser from the force main with quick-connect coupling and appropriate valving shall be provided to hook up portable pumps.

5.3.17.3. Engine-Driven Generating Equipment
Where permanently installed or portable engine-driven generating equipment is used, the following requirements in addition to general requirements shall apply.

5.3.17.3.1. Generating Capacity
Generating unit size shall be adequate to provide power for pump motor starting current and for lighting, ventilation and other auxiliary equipment necessary for safety and proper operation of the pumping station. The operation of only one pump during periods of auxiliary power supply must be justified. Such justification may be made on the basis of maximum anticipated flows relative to single-pump capacity, anticipated length of power outage and storage capacity. Special sequencing controls shall be provided to start pump motors unless the generating equipment has capacity to start all pumps simultaneously with auxiliary equipment operating.

5.3.17.3.2. Operation
Provisions shall 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 5.3.17.2.3.

5.3.17.3.3. Portable Generating Equipment
Where portable generating equipment or manual transfer is provided, sufficient storage capacity to allow time for detection of pump station failure and transportation and connection of generating equipment shall be provided. The use of special electrical connections and double throw switches are recommended for connecting portable generating equipment.

5.3.17.4. Independent Utility Substations
Where independent substations are used for emergency power, each separate substation and its associated transmission lines shall be capable of starting and operating the pump station at its rated capacity.

5.3.18. Instructions and Equipment
The operating authority of sewage pumping stations shall be supplied with a complete set of operational instructions including; emergency procedures, maintenance schedules, tools and such spare parts as may be necessary.

5.3.19. Safety and Housekeeping
Maximum consideration shall be given to providing safe working conditions for the operators; this means, among other things that provision for a high standard of housekeeping shall be essential. Poor housekeeping and poor safety practices in sewage pumping stations cannot be tolerated.

For example, portable ladders, preferably ships' ladders, shall be used rather than manhole ladder rungs. Ships' ladders, stairs with non-slip treads, or mechanical lifts shall be provided for all installations. All electrical wiring shall be properly grounded; lighting shall be adequate in all locations, etc.

5.3.20. Forcemains

5.3.20.1. Diameter and Velocity
In general, forcemains shall be a minimum of 100 mm in diameter. Velocities should be in the range of 0.6 to 2.5 m/s.

5.3.20.2. Termination
Forcemains shall enter the gravity sewer at a point not more than 600 mm above the flow line of the receiving manhole. A 45° bend may be considered to direct the flow downwards.

5.3.20.3. Design Friction Losses

5.3.20.3.1. Friction Coefficient

Friction losses through forcemains shall be based on the Hazen-Williams formula or other acceptable method. When the Hazen-Williams formula is used, the following values for C shall be used for design:

1. Unlined iron or steel: C = 100; and
2. All other: C = 120

5.3.20.3.2. Maximum Power Requirements

When initially installed, forcemains will have a significantly higher "C" factor. The effect of the higher "C" factor should be considered in calculating maximum power requirements and duty cycle time to prevent damage to the motor.

5.3.20.4. Air and Vacuum Relief Valves

Automatic air relief valves shall be placed at all high points in the forcemain to prevent air locking. Vacuum relief valves may be necessary to relieve negative pressures on force mains. The force main configuration and head conditions should be evaluated as to the need for and placement of vacuum relief valves.

5.3.20.5. Pipe and Design Pressure

The forcemain and fittings, including reaction blocking, shall be designed to withstand normal pressure and pressure surges (water hammer) and associated cyclic reversal of stresses that are expected with the cycling of wastewater lift stations. Surge protection chambers should be evaluated.

5.3.20.6. Separation

Watermains and sewage forcemains are to be installed in separate trenches 3 metres apart. The soil between the trenches shall be undisturbed. Forcemains crossing watermains shall be laid to provide a minimum vertical distance of 450 mm between the outside of the forcemain and the outside of the watermain. The watermain shall be above the forcemain. At crossings, one full length of water pipe shall be located so both joints will be as far from the forcemain as possible. Special structural support for the watermain and the forcemain may be required.

5.3.20.7. Slope and Depth

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

5.3.20.8. Identification

Where forcemains are constructed of material, which might cause the forcemain to be confused with potable watermains, the forcemain should be appropriately identified.

5.3.20.9. Testing

Leakage is defined as the amount of water supplied from a water storage tank in order to maintain test pressure for 2 hours. The allowable leakage is 0.03 L/mm pipe diameter per 300 m, per hour for a working pressure of 1000 kPa. For other working pressures, test in accordance with the latest version of AWWA C600.

5.4. Wastewater Treatment

5.4.1. Plant Location

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

1. Proximity to residential areas;
2. Direction of prevailing winds;
3. Accessibility by all-weather roads;
4. Area available for expansion;
5. Protection against flooding;
6. Local zoning requirements;
7. Local soil characteristics, geology, hydrology, and topography available to minimize pumping (selected elevations to maximize gravity flow through the works);
8. Access to receiving stream;
9. Downstream uses of the receiving water body;
10. 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; and
11. Proximity to surface water supplies and water wells.

Where a site must be used which is critical with respect to these items, appropriate measures shall be taken to minimize adverse impacts.

5.4.1.1. Flood Protection

The treatment plant structures, electrical and mechanical equipment shall be protected from physical damage by the 100-year flood. Treatment plants should remain fully operational and accessible during floods. This requirement applies to new construction and to existing facilities undergoing major modification.

5.4.2. Quality of Effluent

The quality of effluent shall meet the *Environmental Control Water and Sewer Regulations, 2003*.

5.4.3. Design

5.4.3.1. Type of Treatment

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

1. Present and future effluent requirements;
2. Location of and local topography of the plant site;
3. Space available for future plant construction;
4. The effects of industrial wastes likely to be encountered;
5. Ultimate disposal of sludge;
6. System capital costs;
7. System operating and maintenance costs, including basic energy requirements;
8. Process complexity governing operating personnel; and
9. Environmental impact on present and future adjacent land use.

5.4.3.2. Required Engineering Data for New Process and Application Evaluation

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

To determine that such new processes and equipment or applications have a reasonable and substantial chance of success, the DOEC may require the following:

1. Adequate monitoring observations, including test results and engineering evaluations, demonstrating the efficiency of such processes under varied flow regimes;

2. Detailed description of the test methods;

3. Testing, including appropriate composite samples, under various ranges of strength and flow rates (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; or

4. Other appropriate information.

The DOEC 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 or developer.

5.4.3.3. Design Loads

5.4.3.3.1. Hydraulic Design

New Systems *- the design for sewage treatment plants to serve new sewage systems shall be based on an average daily flow of 340 L/cap,* unless all season flow data is available, or other justification upon which to better estimate flow is provided.

Existing Systems - where there is an existing system the volume and strength of existing flows shall be determined. The determination shall include both dry-weather and wet-weather conditions. Samples shall be taken and composited so as to be accurately representative of the strength of the wastewater. The following information shall be recorded where appropriate for the design of the works:

1. Peak rates of flow over a sufficient period of time, which would adversely affect the detention time of treatment units or the flow characteristics of the conduits;
2. Percentage of industrial waste flows; and
3. Wet-weather peak flows.

Flow Equalization - facilities for the equalization of flows and organic shock load shall be considered at all plants, which are critically affected by surge loadings.

5.4.3.3.2. Organic Design

Domestic waste treatment design shall be on the basis of at least 0.08 kg of BOD per capita per day and 0.09 kg of suspended solids per capita per day, unless other information is submitted to justify alternate designs.

Domestic waste treatment plants that will receive commercial, institutional and industrial wastewater flows shall be designed to include the waste loads as per Table 5.1.

Septage and leachate may contribute significant organic load and other materials, which can cause operational problems. If septage and/or leachate are to be discharged to the wastewater treatment facility, it must be approved by the DOEC.

When an existing treatment works is to be upgraded or expanded, the organic design shall be based upon the actual strength of the wastewater, with an appropriate increment for growth.

5.4.3.3.3. Minimum Parameters

In the absence of documented data for a specific wastewater treatment facility, the minimum design parameters provided in Sections 5.4.3.3.1 and 5.4.3.3.2 shall be used.

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, shall be considered.

5.4.3.5. Design by Analogy

Data from similar municipalities may be utilized in the design of new systems.

5.4.3.6. Conduits and Piping
All piping and channels should be designed to carry the maximum expected flows. The incoming sewer should be designed for free discharge. Bottom corners of the channels must be filleted. Pockets and corners where solids can accumulate should be eliminated. 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 shall be used for these control gates. Pipe and accessories used for conduits shall conform to the standard specifications of the AWWA or equivalent. In general, the following factors shall be considered in designing conduits: carrying capacity, maximum pressures, present and future water hammer, hydraulic grade, traffic loads, laying conditions, expansion and contraction, anchorage at bends and joints, and depth of cover.

5.4.3.7. Arrangement of Units
Component parts of the plant should be arranged for greatest operating and maintenance convenience, flexibility, economy continuity of maximum effluent quality, and ease of installation of future units.

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

5.4.4. Plant Details

5.4.4.1. Installation and Inspection of Mechanical Equipment
The specifications should be so written that the manufacturer would commission the installation and initial operation of major items of mechanical equipment, and provide appropriate training to the operator.

5.4.4.2. Unit Dewatering, Flotation Protection and Plugging
Means such as drains or sumps shall be provided to dewater each unit to an appropriate point in the process as per manufacturer's recommendations. Due consideration shall be given to the possible need for hydrostatic pressure relief devices to prevent flotation of structures. Pipes subject to plugging shall be provided with means for mechanical cleaning or flushing.

5.4.4.3. Unit Bypasses
5.4.4.3.1. Removal from Service
Properly located and arranged bypass structures shall be provided so that each unit of the plant can be removed from service independently. The bypass design shall facilitate plant operation during unit maintenance and emergency repair so as to minimize deterioration of effluent quality and insure rapid process recovery upon return to normal operational mode.

5.4.4.3.2. Unit Bypass During Construction

A plan for the method and level of treatment to be achieved during construction shall be developed and included in the facility plan that must be submitted to the DOEC for review and approval. This approved treatment plan must be implemented by inclusion in the plans and specifications to be bid for the project.

5.4.4.4. Construction Materials

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

5.4.4.5. Painting

In order to facilitate identification of piping, particularly in large plants, it is suggested that the different lines have contrasting colours. The colour code, as per Table 5.5, is recommended for purposes of standardization for all sewage systems in the province:

Table 5.5
Paint Colour Code for Sewage Systems

Contents	Colour Scheme
Raw Sludge line	Brown w/ black bands
Sludge Recirculation suction line	Brown w/ yellow bands
Sludge draw off line	Brown w/ orange bands
Sludge Recirculation discharge line	Brown
Sludge gas line	Orange(or red)
Natural Gas line	Orange(or red) w/ black bands
Potable water line	Blue
Non-potable water system	Blue with Black bands
Chlorine line	Yellow
Sulphur Dioxide	Yellow w/ red bands
Sewage (wastewater) line	Grey
Compressed air line	Green
Water lines for heating digesters or buildings	Blue w/ 150mm red band spaced 750mm apart

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

5.4.4.6. Operating Equipment
The specifications should include a complete outfit of tools, accessories for the plant operator's use, such as squeegees, wrenches, valve keys, rakes, shovels, spare parts, etc. A portable pump is desirable. Readily accessible storage space and workbench facilities shall be provided and consideration shall be given to provision of a garage for large equipment storage, maintenance and repair.

5.4.4.7. Grading and 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 shall not be permitted to drain into any unit. Particular care shall be taken to protect trickling filter beds, sludge beds, and intermittent sand filters, from surface water. Provision should be made for attractive landscaping, particularly when a plant is to be located close to residential areas.

5.4.4.8. Erosion Control During Construction
Effective site erosion control shall be provided during construction.

5.4.5. Plant Outfalls
Refer to Section 5.2.15.

5.4.6. Essential Facilities

5.4.6.1. Emergency Power Facilities
A standby source of power shall be provided where the discharge of raw or partially treated sewage may endanger public health or cause damage to the environment and where the treatment process or biomass may be adversely affected due to oxygen depletion and septic conditions. The need for standby power and the extent of equipment requiring operation by standby power must be individually assessed for each sewage treatment plant. Some of the factors which will require consideration in making the decisions regarding standby power and the processes to be operated by the standby power equipment are as follows:

1. Reliability of primary power source;

2. Number of power feeder lines supplying grid system, number of alternate routes within the grid system, and the number of alternate transformers through which power could be directed to the sewage treatment plant;

3. Whether sewage enters the plant by gravity or is pumped;

4. Type of treatment provided;

5. Pieces of equipment which may become damaged or overloaded following prolonged power failure;

6. Assimilation capacity of the receiving waters and ability to withstand higher pollution loadings over short time periods; and

7. Other uses of the receiving water.

Each specific installation should provide for the following considerations:
1. Means for illuminating working areas to ensure safe working conditions; and

2. Standby power source or equivalent to power pumps, motorized valves and control panels that are necessary to maintain the sewage flow through the treatment plant.

5.4.6.1.1. Power for Aeration

Standby generating capacity normally is not required for aeration equipment used in the activated sludge process. In cases where a history of long-term (4 hours or more) power outages has occurred, auxiliary power for minimum aeration of the activated sludge will be required. Full power generating capacity may be required by the reviewing authority on certain critical stream segments.

5.4.6.1.2. Power for Disinfection

Continuous disinfection, where required, shall be provided during all power outages. Continuous dechlorination is required for all systems using chlorine.

5.4.6.2. Measurement of Flow

5.4.6.2.1. Facilities

Flow measurement facilities shall be provided at all plants. Indicating, totalizing and recording flow measurement devices shall be provided for all treatment plants. All flow measurement equipment must be sized to function effectively over the full range of flows expected and shall be protected against freezing.

5.4.6.2.2. Location

Consideration shall be given to the provision of automatic samplers at large plants.

Flow measurement facilities shall be provided to measure the following flows:
1. Plant influent or effluent flow;
2. Plant influent flow, if influent flow is significantly different from effluent flow, both shall be measured. This would apply for installations such as lagoons, sequencing batch reactors and plants with excess flow storage or flow equalization;
3. Excess flow treatment facility discharges;
4. Other flows required to be monitored under the provisions of the Permit to Operate; and
5. Other flows such as return activated sludge, waste activated sludge, recirculation and recycle required for plant operational control.

5.4.6.2.3. Hydraulic Conditions

Flow measurement equipment including approach and discharge conduit configuration and critical control elevations shall be designed to ensure that the required hydraulic conditions necessary for accurate measurement are provided. Conditions that must be avoided include turbulence, eddy currents, and air entrainment

5.4.6.3. Septicity

Where it can be expected that the raw sewage will be largely devoid of dissolved oxygen, provisions should be made for aeration, recirculation or other means of offsetting septic action and odour nuisance and to ensure efficient operation of the plant.

5.4.6.4. Water Supply

An adequate supply of potable water, under pressure shall be provided for sanitary and drinking purposes, use in the laboratory and for general cleanliness around the plant. All plumbing shall comply with the requirements of the Canadian Plumbing Code. No piping or other connections shall exist in any part of the treatment works, which, under any conditions, might cause the contamination of a potable water supply. The chemical quality should be checked for suitability for its intended use such as in heat exchangers, chlorinators, etc.

The municipal water system and the potable water piping system within the treatment building shall be protected from the potential backflow of sewage due to back-siphonage or backpressure with the use of certified backflow prevention devices. In this case, the minimum device(s) to be used on the building water service line(s) shall be a reduced pressure zone backflow preventer, meeting or exceeding CSA 64.4. A certified tester must test this device on an annual basis. For protection of water users within the building, appropriate vacuum breakers, as determined by the Engineer, shall be installed on any threaded water connection. There shall be no connection made to the potable water supply within the treatment building between the reduced pressure zone backflow preventer and the street source.

5.4.6.4.1. 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:

1. Lavatory;
2. Water closet;
3. Laboratory sink (with vacuum breaker);
4. Shower
5. Drinking fountain;
6. Eye wash fountain; and
7. Safety shower.

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

5.4.6.4.2. Indirect Connections

Where a potable water supply is to be used for any purpose in a plant other than those listed in Section 5.4.6.4.1, a break tank, pressure pump and pressure tank shall be provided. Water shall be discharged to the 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 shall be permanently posted at every hose bibb, faucet, hydrant or sill cock located on the water system beyond the break tank to indicate that the water is not safe for drinking.

5.4.6.4.3. Separate Potable Water Supply

Where it is not possible to provide water from a public water supply, a separate well may be provided. Location and construction of the well should comply with requirements and regulations of the DOEC. Requirements governing the use of the supply are those contained in Sections 5.4.6.4.1 and 5.4.6.4.2.

5.4.6.4.4. 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 system outlets shall be posted with a permanent sign indicating the water is not safe for drinking.

5.4.6.5. Sanitary Facilities

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

Floor surfaces shall be sloped adequately to a point of drainage.

5.4.6.6. Stairways

Stairways shall be installed in compliance with the National Building Code.

5.4.6.7. Instrumentation and Control

Refer to Section 6.0 on Instrumentation and Control.

5.4.6.8. Laboratory Facilities

Laboratory facilities, suitable for controlling the operations and determining the efficiency of the various treatment units, shall be provided at every plant. A list of laboratory supplies and equipment shall be included in the specifications or a special arrangement such as a cash allowance to cover the specific laboratory equipment needed. The plans shall include sufficient laboratory space and appurtenances in the control building or elsewhere. Specific requirements should be obtained from the DOEC.

5.4.7. Safety

Adequate provision shall be made in accordance with the Occupational Health and Safety requirements to effectively protect the operator and visitors from hazards. The following shall be provided to fulfil the particular needs of each plant:

1. Enclosure of the plant site with a fence and signs designed to discourage the entrance of unauthorized persons and animals;

2. Hand rails and guards around tanks, trenches, pits, stairwells, and other hazardous structures with the tops of walls less than 1 m above the surrounding ground level;

3. Gratings over appropriate areas of treatment units where access for maintenance is required;

4. First aid equipment;

5. “No Smoking” signs in hazardous areas;

6. Protective clothing and equipment, such as self-contained breathing apparatus, gas detection and oxygen depletion equipment, goggles, gloves, hard hats, safety harnesses, eye wash stations, stations, etc.;

7. Potable blower and sufficient hose;

8. Portable lighting equipment complying with the National and Provincial Electrical Code requirements;

9. Gas detectors;

10. 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, etc.;

11. Adequate ventilation in pump station areas;

12. Provisions for local lockout on stop motor controls; and

13. Provisions for confined space entry.

In addition, reference should be made to appropriate federal and provincial legislation regarding hazardous chemicals.

5.5. Screening And Grit Removal

5.5.1. Bar Racks and Screens

Coarse bar racks or screens shall be provided as the first treatment stage for the protection of plant equipment against reduced operating efficiency, blockage, or physical damage.

5.5.1.1. Selection Considerations

When considering which types of screening devices should be used, the following factors should be considered:

1. Effect on downstream treatment and sludge disposal operations;
2. Possible damage to comminutor or barminutor devices caused by stones or coarse grit patches;
3. Head losses of the various alternative screening devices;
4. Maintenance requirements;
5. Screenings disposal requirements, and quantities of screenings; and
6. Requirements for a standby unit.

5.5.1.2. Access

Appropriate access provisions should be provided as per appropriate Occupational Health and Safety requirements

5.5.1.3. Ventilation

Appropriate ventilation provisions should be provided as per appropriate Occupational Health and Safety requirements.

5.5.1.4. Design

5.5.1.4.1. Velocity

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 shall 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.

5.5.1.4.2. Bar Spacing

1. Manually Cleaned Screens - Clear openings between bars should be from 25 mm to 45 mm. Design and installation shall be such that they can be conveniently cleaned.

2. 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.

5.5.1.4.3. Slope

Manually cleaned screens, except those for emergency use, should be placed on a slope of 30 to 45 degrees with the horizontal.

5.5.1.4.4. Channels
Dual channels shall be provided and equipped with the necessary gates to isolate flow from any screening unit. Provisions shall be made to facilitate dewatering each unit. The channel preceding and following the screen shall be shaped to eliminate standing and settling of solids.

5.5.1.4.5. Materials
Materials for bar screens shall depend on the type of sewage. All components subject to wear, which come in contact with the wastewater, shall be made from 316 stainless steel.

5.5.1.4.6. Lighting
Provide adequate lighting in the screening chamber.

5.5.1.5. Safety
5.5.1.5.1. Railings and Gratings
Manually cleaned screen channels shall be protected by guard railings and deck gratings, with adequate provisions for removal or opening to facilitate raking. Mechanically cleaned screen channels shall also be protected by guard railings and deck gratings. Consideration should also be given to temporary access arrangements to facilitate maintenance and repair.

5.5.1.5.2. Mechanical Devices
Mechanical screening equipment shall 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 shall be provided.

5.5.1.5.3. Drainage
Floor design and drainage shall be provided to prevent slippery areas.

5.5.1.6. Control Systems
5.5.1.6.1. 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 preset high water elevation. If the cleaning mechanism fails to lower the high water, a warning should be signalled.

5.5.1.6.2. Electrical Systems and Components
Electrical systems and components (i.e. motors, lights, cables, conduits, switchboxes, control circuits, etc.) in enclosed or partially enclosed spaces where flammable mixtures occasionally may be present (including all space above raw or partially treated wastewater) shall comply with the Canadian Electrical Code, Part 1 and the regulations under the applicable Provincial Power Standards. All electrical components in the headworks room must be explosion proof.

5.5.1.6.3. Manual Override
Automatic controls shall be supplemented by a manual override.

5.5.1.7. Screenings Removal and Disposal
Adequate means for removing screenings shall be provided. Screw conveyor equipment may be necessary depending on the depth of pit and amount of screenings or equipment to be lifted.

Facilities must be provided for handling, storage, and disposal of screenings in a manner acceptable to the DOEC.

Manually cleaned screening facilities shall include an accessible platform from which the operator may rake screenings easily and safely. Suitable drainage facilities shall be provided for both the platform and storage area.

5.5.1.8. Auxiliary Screens
Where mechanically operated screening or comminuting devices are used, auxiliary manually cleaned screens shall be provided. Where two or more mechanically cleaned screens are used, the design shall provide for taking any unit out of service without sacrificing the capability to handle the peak design flow.

5.5.1.9. Fine Screens
Fine screens may be used in lieu of primary sedimentation providing that subsequent treatment units are designed on the basis of anticipated screen performance. Fine screens are not to be considered equivalent to primary sedimentation

5.5.2. Comminutors
Comminutors or grinders shall 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.

5.5.2.1. Location
Where possible comminutors should be located downstream of any grit removal equipment and be protected by a coarse screening device. Provisions for location shall be in accordance with those for screening devices.

5.5.2.2. Size
The comminutor shall be sized to handle design peak hourly flow.

5.5.2.3. Installation
1. A screened bypass channel shall be provided. The use of the bypass channel should be automatic at depths of flow exceeding the design capacity of the comminutor.

2. A 150 mm deep gravel trap should protect each comminutor that is not preceded by grit removal equipment.

3. Gates shall be provided in accordance with Section 5.5.1.4.4.

4. The loss of pressure through comminutors should be minimized.

5.5.2.4. Servicing

Provision shall be made to facilitate servicing units in place and removing units from their location for servicing.

5.5.2.5. Electrical Controls and Motors

Electrical equipment in comminutor chambers where hazardous gases may accumulate shall comply with the Canadian Electrical Code and applicable Provincial Power Standards. Motors in areas not governed by this requirement may need protection against accidental submergence.

5.5.3. 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 sewage treatment plants and are required for plants receiving sewage 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 shall include provisions for future installation. Consideration shall 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.

5.5.3.1. Location

Grit removal facilities should be located ahead of pumps and comminuting devices. Coarse bar racks should be placed ahead of grit removal facilities.

5.5.3.2. 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.

5.5.3.3. Ventilation

Where installed indoor, uncontaminated air shall be introduced continuously at a rate of 12 air changes per hour, or intermittently at a rate of 30 air changes per hour. Odour control facilities may also be warranted.

5.5.3.4. Electrical

Electrical equipment in grit removal areas where hazardous gases may accumulate shall comply with the Canadian Electrical Code and the regulations under the applicable provincial Power Standards.

5.5.3.5. Outside Facilities

Grit removal facilities located outside shall be protected from freezing.

5.5.3.6. Design Factors

5.5.3.6.1. Inlet

Inlet turbulence shall be minimized.

5.5.3.6.2. Type and Number of Units

Grit removal facilities (channel type) should have at least two hand-cleaned, or a mechanically cleaned unit with bypass. A single manually cleaned or mechanically cleaned grit chamber with bypass is acceptable for small sewage treatment plants serving separate sanitary sewer systems. Minimum facilities for larger plants serving separate sanitary sewers should be at least one mechanically cleaned unit with a bypass. Facilities other than channel-type are acceptable if provided with adequate and flexible controls for agitation and/or air supply devices and with grit collection and removal equipment.

5.5.3.6.3. Grit Channels

5.5.3.6.3.1. Velocity

Channel-type chambers shall be designed to provide controlled velocities as close as possible to 0.30 m/s for normal variation in flow.

5.5.3.6.3.2. Control Sections

Flow control sections shall be of the proportional or Sutro Weir type.

5.5.3.6.3.3. Channel Dimensions

The minimum channel width shall be 375 mm. The minimum channel length shall 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.

5.5.3.6.4. 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.

5.5.3.6.5. 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 flow rate. 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:

1. 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;

2. A classifying-type conveyor can be used to remove the grit and return the organics to the detritus tank; or

3. The removed detritus can be passed through a centrifugal-type separator.

5.5.3.6.6. 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 parameters in the following sub-sections.

5.5.3.6.6.1. Detention Time
Detention time shall be 2 to 5 minutes at the peak sewage flow rate.

5.5.3.6.6.2. Air Supply
Air supply rates should be in the range of 4.5 to 12.4 L/s per linear meter of tank. The higher rates should be used with tanks of large cross-section (i.e. greater than 3.6 m deep). 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.

5.5.3.6.6.3. Inlet Conditions
Inlet flow should be parallel to induced roll in tank. There shall be a smooth transition from inlet to circulation flow.

5.5.3.6.6.4. Baffling
A minimum of one transverse baffle near the outlet weir shall be provided. Additional transverse baffles in long tanks and longitudinal baffles in wide tanks should be considered.

5.5.3.6.6.5. Outlet Conditions
The outlet weir shall be oriented parallel to the direction of induced roll (i.e. at a right angle to the inlet).

5.5.3.6.6.6. Tank Dimensions
The lower limit of the above aeration rates are generally suitable for tanks up to 3.7 m deep and 4.3 m wide. Wider or deeper tanks require aeration rates in the upper end of the above range. Long, narrow aerated grit tanks are generally more efficient than short tanks and produce a cleaner grit. A length to width ratio of 2:5 to 5:1 is desirable. Depth to width ratios of 1:1.5 to 1:2 is acceptable.

5.5.3.6.6.7. 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 per cent of above). The velocity across the floor of the tank shall not be less than 0.3 m/s.

5.5.3.6.6.8. 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.

5.5.3.6.6.9. 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).

5.5.3.6.7. Mechanical Grit Chambers

Specific design parameters for mechanical grit chambers will be evaluated on a case-by-case basis.

5.5.3.6.8. Grit Washing

The need for grit washing should be determined by the method of final grit disposal.

5.5.3.6.9. Dewatering

Provision shall be made for isolating and dewatering each unit. The design shall provide for complete draining and cleaning by means of a sloped bottom equipped with a drain sump.

5.5.3.6.10. Water

An adequate supply of water under pressure shall be provided for cleanup.

5.5.3.7. 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 manlift, adequate ventilation and adequate lighting.

5.5.3.8. 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 shall be provided for grit handling areas. Grit transporting facilities shall be provided with protection against freezing and loss of material.

5.5.3.9. Grit Disposal

Disposal of grit shall be in accordance to the requirements of the DOEC.

5.5.4. Pre-aeration and Flocculation

Pre-aeration of raw wastewater, may be used to achieve one or more of the following objectives:

1. Odour control;
2. Grease separation and increased grit removal;
3. Prevention of septicity;
4. Grit separation;
5. Flocculation of solids;
6. Maintenance of DO in primary treatment tanks at low flows;
7. Increased removals of BOD and SS in primary units; and
8. Minimizes solids deposits on sidewalls and bottom of wet wells.

Flocculation of sewage with or without coagulating aids is worthy of consideration when it is desired to reduce the strength of sewage prior to subsequent treatment. Also, flocculation may be beneficial in pre-treating sewage containing certain industrial wastes.

5.5.4.1. Arrangement

The units should be designed so that removal from service will not interfere with normal operation of the remainder of the plant.

5.5.4.2. Pre-aeration

Figure 5.1 represents airflow 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.

5.5.4.3. Flocculation

5.5.4.3.1. Detention Period

When air or mechanical agitation is used in conjunction with chemicals to coagulate or flocculate the sewage, the detention period should be about 30 minutes at the design flow. However, if polymers are used this may be varied.

5.5.4.3.2. Stirring Devices

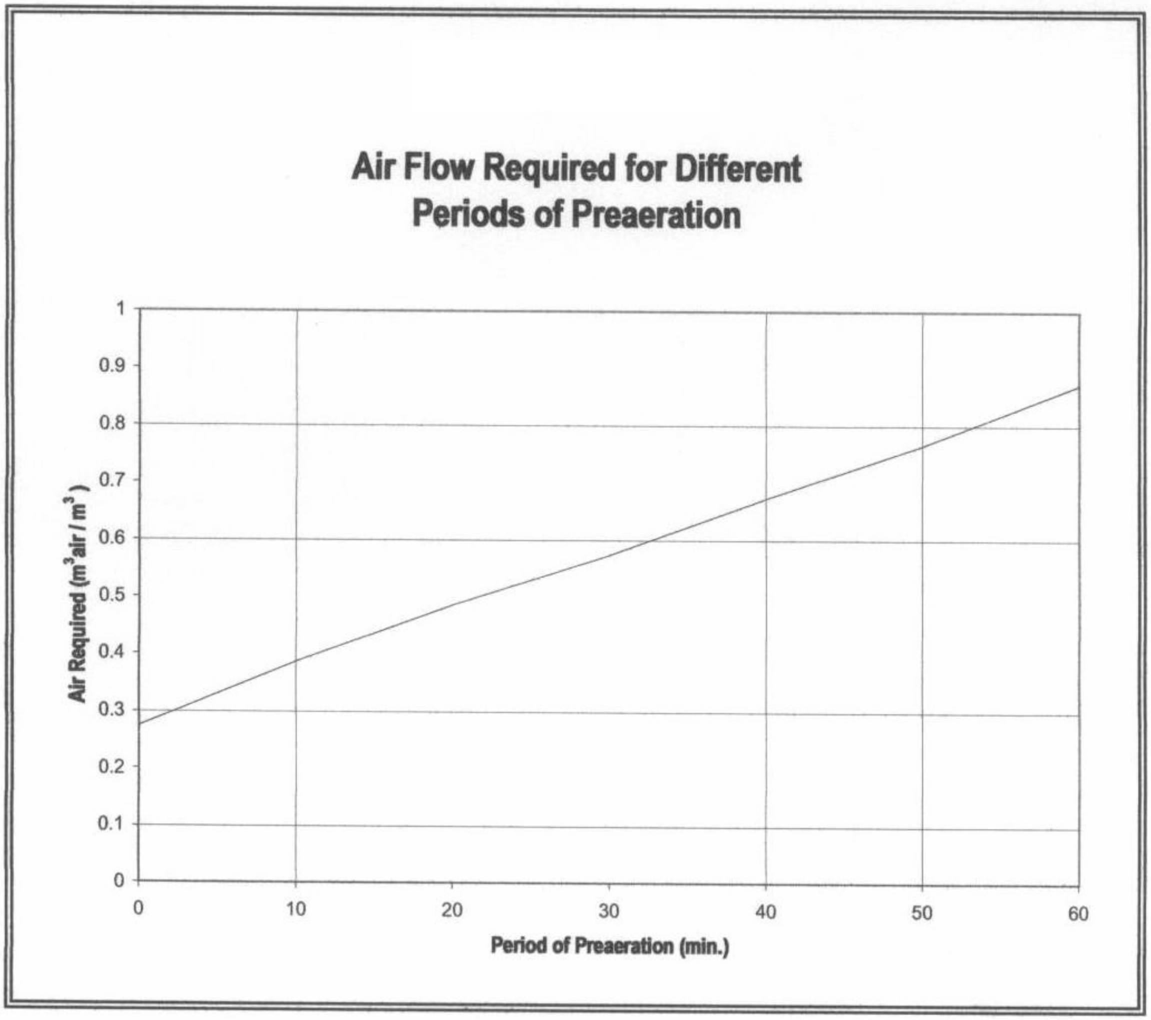
5.5.4.3.2.1. Paddles

Paddles should have a peripheral speed of 0.50 to 0.75 m/s to prevent deposition of solids.

5.5.4.3.2.2. Aerators

Any of the types of equipment used for aerating activated sludge may be utilized. It shall be possible to control agitation, to obtain good mixing and maintain self-cleaning velocities across the tank floor.

Figure 5.1



5.5.4.3.3. Details

Inlet and outlet devices should be designed to insure proper distribution and to prevent short-circuiting. Convenient means should be provided for removing grit.

5.5.4.3.4. Rapid Mix

At plants where there are two or more flocculation basins utilizing chemicals, provision shall be made for a rapid mix of the sewage with the chemical so that the sewage 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 3.0 minutes.

5.5.5. Flow Equalization

Flow equalization can reduce the dry-weather variations in organic and hydraulic loadings at any wastewater treatment plant. It should be provided where large diurnal variations are expected.

5.5.5.1. Location

Equalization basins should be located downstream of pre-treatment facilities such as bar screens, comminutors and grit chambers.

5.5.5.2. Type

Flow equalization can be provided by using separate basins or on-line 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.

5.5.5.3. 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:

1. 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.

2. Draw parallel lines to the first line through the points on the curve farthest from the first line; and

3. Draw a vertical line between the lines drawn in No. 2. The length of this line represents the minimum required volume.

5.5.5.4. Operation

5.5.5.4.1. Mixing

Where applicable, aeration or mechanical equipment shall 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.

5.5.5.4.2. Aeration

Where applicable, aeration equipment shall be sufficient to maintain a minimum of 1.0 mg/L of dissolved oxygen in the mixed basin contents at all times. Air supply rates should be a minimum of 0.15 L/s per cubic meter 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.

5.5.5.4.3. Controls
Inlets and outlets for all basin compartments shall 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 shall also be provided to measure and indicate liquid levels and flow rates.

5.5.5.5. Electrical
All electrical work in housed equalization basins shall comply with the Canadian Electrical Code and the regulations under applicable Provincial Power Standards.

5.5.5.6. Access
Suitable access shall be provided to facilitate cleaning and the maintenance of equipment.

5.6. Clarification

5.6.1. Sedimentation Tanks

5.6.1.1. Design Requirements
The need for and the design of primary sedimentation tanks will be influenced by various factors, including the following:

1. 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 sludges);

2. The presence, or absence, of secondary treatment following primary treatment;

3. The need for handling of waste activated sludge in the primary settling tank; and

4. The need for, or possible economic benefits through, phosphorus removal in the primary settling tank(s).

5.6.1.1.1. Number of Units
Multiple units capable of independent operation are desirable and shall be provided in all plants where design flows exceed 500 m^3/d. Plants not having multiple units shall include other provisions to assure continuity of treatment.

5.6.1.1.2. Arrangement of Units
Settling tanks shall be arranged in accordance with Section 5.4.3.7.

5.6.1.1.3. Interaction with Other Processes
1. 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.

2. For activated sludge plants employing high-energy aeration, provisions should be made for floc to be reformed before settling.

3. 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 waste activated sludge.

5.6.1.1.4. Flow Distribution and Control

Effective flow measurement devices and control appurtenances (i,e., valves, gates, splitter boxes, etc.) shall be provided to permit proper proportion of flow to each unit. Parallel basins should be of the same size, otherwise flow shall be distributed in proportion to surface area.

5.6.1.1.5. Tank Configuration and 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 4:1, and width to depth ratios of 1:1 and 2.25:1.

5.6.1.1.6. Site Constraints

The selection of feasible clarifier alternatives should include the following site considerations:
1. Wind direction;
2. Proximity to residents;
3. Soil conditions
4. Groundwater conditions; and
5. Available space.

5.6.1.1.7. Size Limitations

Rectangular clarifiers shall have a maximum length of 90 m. Circular clarifiers shall have a maximum diameter of 60 m. The minimum length of flow from inlet to outlet shall be 3 m, unless special provisions are made to prevent short-circuiting. The vertical sidewater depth shall be designed to provide an adequate separation zone between the sludge blanket and the overflow weirs.

5.6.1.1.8. Inlet Structures

Inlet structures should be designed to dissipate the inlet velocity, to distribute the flow equally and to prevent short-circuiting. Channels should be designed to maintain a velocity of at least 0.3 m/s at one-half design average flow. Corner pockets and dead ends should be eliminated and corner fillets or channelling used where necessary. Provisions shall be made for elimination or removal of floating materials in inlet structures.

5.6.1.1.9. Outlet Arrangements

Overflow weirs shall be adjustable for levelling, and sufficiently long to avoid high heads, which result in updraft currents.

5.6.1.1.9.1. Location

Overflow weirs shall be located to optimize actual hydraulic detention time and minimize short-circuiting. Peripheral weirs shall be placed at least 0.3 m from the wall.

5.6.1.1.9.2. Weir Troughs

Weir troughs shall be designed to prevent submergence at maximum design flow and to maintain a velocity of at least 0.3 m/s at one-half design average flow.

5.6.1.1.10. Submerged Surfaces

The tops of troughs, beams and similar submerged construction elements shall have a minimum slope of 1.4 vertical to 1 horizontal; the underside of such elements should have a slope of 1:1 to prevent the accumulation of scum and solids.

5.6.1.1.11. Unit Dewatering

Unit dewatering features shall conform to the provisions outlined in Section 5.4.4.2 (Unit Dewatering, Flotation Protection and Plugging). The bypass design should also provide for redistribution of the plant flow to the remaining units.

5.6.1.1.12. Freeboard

Walls of settling tanks shall extend at least 150 mm above the surrounding ground surface and shall 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 or excessive heat loss resulting in freezing problems.

5.6.1.1.13. Clarifier Covers

Clarifiers may be required to be covered for winter operation. The structure should be constructed with adequate headroom for easy access. The structure must include adequate lighting, ventilation and heating. Humidity and condensation shall be controlled inside the structure.

5.6.1.1.14. Surface Settling Rates

5.6.1.1.14.1. Primary Settling Tanks

Units should be designed for a detention period of 1.0 hour at peak flow and a surface settling rate of approximately 61 $m^3/(m^2.d)$ at peak flows, assuming activated sludge wasting followed by aeration. Other rates and detention periods shall be considered to meet special conditions such as dictated by the secondary treatment process, character of waste, or to adjust the efficiency to meet specific requirements. Primary settling of normal domestic sewage can be expected to remove 30 to 35% of the influent BOD. However, anticipated BOD removal for sewage containing appreciable quantities of industrial wastes (or chemical additions to be used) should be determined by laboratory tests and consideration of the quantity and character of the wastes.

5.6.1.1.14.2. Intermediate Settling Tanks

Surface settling rates for intermediate settling tanks following series units of fixed film reactor processes should not exceed 61 $m^3/(m^2.d)$ at peak flow.

5.6.1.1.14.3. Final Settling Tanks

Settling tests should be conducted where a pilot study of biological treatment is warranted by unusual waste characteristics or treatment requirements. Testing shall be done where proposed loadings exceed the limits sets forth in this section:

1. **Fixed Film Biological Reactors**

   Surface settling rates for settling tanks following trickling filters or rotating biological contactors should not exceed 49 $m^3/(m^2.d)$ at peak flow.

2. **Activated Sludge**

   The hydraulic loadings should not exceed 49 $m^3/(m^2.d)$ at peak flow with no phosphorus removal and 40 $m^3/(m^2.d)$ at peak flow with phosphorus removal. The solids loading for all activated sludge processes should not exceed 244 $kg/(m^2.d)$ at the peak flows. Consideration should be given to flow equalization

5.6.1.2. Types Of Settling

5.6.1.2.1. 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. Because each particle is assumed to settle independently and with a constant velocity, a mathematical development is possible, based on Newton's Law and Stoke's Law.

5.6.1.2.2. Type II Settling (Flocculant Settling)

Type II settling occurs when particles initially settle independently but flocculate as they proceed the depth 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.

5.6.1.2.3. Type III Settling (Hindered or 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 flocculant processes. As a result, the contacting particles tend to settle as a zone or blanket, and maintain the same position relative to each other.

5.6.1.2.4. Settling Type Design Criteria

Table 5.6 outlines design parameters for sedimentation tanks based upon their associated settling type.

Table 5.6
Sedimentation Basins Design Parameters

Type of Settling	Sidewater Depth (m)	Surface Overflow (1) Rate (m^3/m^2.d)	Weir Loading Rate (m^3/m.d)(1)(2)	Solids Loading (kg/m^2d)
Type I and II	3.0 – 4.6	≤ 40 at design average flow ≤ 60 at peak hourly flow	125 – 370	N/A
Type III	3.5 – 4.6	**1) Settling following activated sludge:** ≤ 30 at design average flow ≤ 50 at peak hourly flow **2) Settling following extended aeration:** ≤ 15 at design average flow ≤ 35 at peak hourly flow **3) Settling following fixed film processes:** ≤ 25 at design average flow ≤ 45 at peak hourly flow **4) Settling following separate nitrification:** ≤ 35 at peak hourly flow	< 250(4)	49 (at SVI of 300) -290 (at SVI of 100)

(1)When several different overflow criteria are given (design average flow, peak hourly flow) the clarifier area to be used in the design is the larger of those computed in each case.
(2)At design average flow.
(3)If pumping is required, weir loading rate should be related to pump delivery rates to avoid short-circuiting.
(4)Where weirs are located so that density currents upturn below them, the rate should not exceed 186 m^3/m.d.

5.6.1.3. Scum and Sludge Removal

5.6.1.3.1. Scum Removal

Effective scum collection and removal facilities, including baffling, shall 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.

5.6.1.3.2. Sludge Removal

Sludge collection and withdrawal facilities shall 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 shall have its own sludge withdrawal lines to insure adequate control of the sludge wasting rate for each tank.

5.6.1.3.2.1. Sludge Collection

Sludge collection mechanisms shall remain in operation during sludge withdrawal. Mechanism speeds shall be such as to avoid undue agitation while still producing desired collection results.

5.6.1.3.2.2. Sludge Hopper

The minimum slope of the sidewalls shall be 1.7 vertical to 1 horizontal. Hopper wall surfaces should be made smooth with rounded corners to aid in sludge removal. Hopper bottoms shall 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.

5.6.1.3.2.3. Cross-Collectors

Cross-collectors, serving one or more settling tanks, may be useful in place of multiple sludge hoppers.

5.6.1.3.2.4. Sludge Removal Piping

Each hopper shall have an individually valved sludge withdrawal line at least 150 mm in diameter. The static head available for withdrawal of sludge shall be 750 mm or greater, as necessary to maintain a 1.0 m/s velocity in the withdrawal line. Clearance between the end of the withdrawal line and the hopper walls shall be sufficient to prevent 'bridging" of the sludge. Adequate provisions shall be made for rodding or back-flushing individual pipe runs. Piping shall also be provided to return waste sludge to primary clarifiers.

5.6.1.3.2.5. Sludge Removal Control

Sludge wells equipped with telescoping valves or other appropriate equipment shall 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 shall be provided. Air lift type of sludge removal will not be approved for removal of primary sludge. Sludge pump motor control systems shall include time clocks and valve activators for regulating the duration and sequencing of sludge removal.

5.6.2. Enhanced Primary Clarification

5.6.2.1. 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 total suspended solids (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. Additional information on the selection and application of chemicals for phosphorus removal is included in Section 5.14 dealing with tertiary treatment and nutrient 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.

5.6.2.1.1. Chemical Coagulants.

Historically, iron salts, aluminum salts, and lime have been the chemical coagulants used for wastewater treatment. Iron salts have typically been the most common of the coagulants used for primary treatment. 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. Organic polymers are sometimes used as flocculation aids.

5.6.2.1.2. 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 over-mixing or under-mixing. Over-mixing may reduce the removal efficiency by breaking up existing wastewater solids and newly formed floc. Under-mixing 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 1000 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 flow rate, 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 flow rate.

5.6.2.1.3. 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 occur. 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.

5.6.2.1.4. 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.

5.6.2.2. Plate and 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 through the use of 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.

5.6.2.2.1. 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 (θ) from the horizontal will have an equivalent settling area which can be calculated utilizing the equation:

$$Total\,Settling\;Area = n\,A\left(\cos\theta\right)$$

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.

5.6.2.2.2. Configuration

Typical settling plates are approximately 0.2 - 0.6 m wide and 3 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 break-up.

5.6.2.3. 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:

1. Coagulation;
2. Injection;
3. Maturation; and
4. 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 microns. 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/d.

5.6.3. Dissolved Air Flotation

Dissolved air flotation (DAF) 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, meatpacking, 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 of biological, mining, and metallurgical processes.

5.6.3.1. Process Design Considerations and 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 into the recycle. 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 and 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:

1. Type and characteristics of feed solids;
2. Hydraulic loading rate;
3. Solids-loading rate;
4. Air-to-solids ratio;
5. Chemical conditioning;
6. Operating policy;
7. Float-solids concentration; and
8. Effluent clarity.

5.6.3.1.1. 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 SVI 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.

5.6.3.1.2. Hydraulic Loading Rate

Hydraulic loading rate refers to the sum of the feed and recycle flow rates divided by the net available flotation area. Dissolved air flotation clarifiers typically are designed for hydraulic loading rates of 60 to 120 m/d, assuming no use of conditioning chemicals. The additional turbulence in flotators when the hourly hydraulic loading rate exceeds 5 m/h 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 hydraulic loading rates greater than 5 m/h.

5.6.3.1.3. 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^{2}h. The difficulties generally are caused by coincidental operation of excessive hydraulic loading rates 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^{2}h 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.

5.6.3.1.4. Feed-Solids Concentration

Changes in feed-solids concentration indirectly affect flotation in connection with the resultant changes in operating conditions. If the fed flow rate, 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 many be required when operating strategy includes maintenance of a specific float-blanket depth or range of depths.

5.6.3.1.5. 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 0.01:1 to 0.4:1; adequate flotation is achieved in most municipal wastewater clarification applications at ratios of 0.02:1 to 0.06:1. 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.

Because 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%.

5.6.3.1.6. 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.

5.6.3.1.7. 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.

5.6.4. Protective and Service Facilities

5.6.4.1. Operator Protection

All clarification tanks shall be equipped to enhance safety for operators. Such features shall appropriately include machinery covers, lifelines, stairways, walkways, handrails and slip-resistant surfaces.

5.6.4.2. Mechanical Maintenance Access

The design shall provide for convenient and safe access to routine maintenance items such as gearboxes, scum removal mechanisms, baffles, weirs, inlet stilling baffle area, and effluent channels.

5.6.4.3. Electrical Fixtures and Controls

Electrical fixtures and controls in enclosed settling basins shall comply with the Canadian Electrical Code and the applicable Provincial Power Standards. The fixtures and controls shall be located so as to provide convenient and safe access for operation and maintenance. Adequate area lighting shall be provided.

5.7. Sludge Handling and Disposal

Sludge handling and disposal must be considered as an integral part of any complete sewage treatment system. The following is a summary of the sludge handling and disposal options and the various process and treatment requirements best suited to the option selected. Re-use and recovery alternatives of sludge by-products are also included as disposal options.

Plans and specifications for sludge handling disposal must be incorporated in the design of all sewage treatment facilities.

5.7.1. Process Selection

The selection of sludge handling unit processes should be based upon at least the following considerations:

1. Local land use;
2. System energy requirements;
3. Cost effectiveness of sludge thickening and dewatering;
4. Equipment complexity and staffing requirements;
5. Adverse effects of heavy metals and other sludge components upon the unit processes;
6. Sludge digestion or stabilization requirements;
7. Side streams or return flow treatment requirements (e.g. digester or sludge storage facilities supernatant, dewatering unit filtrate, wet oxidation return flows);
8. Sludge storage requirements;
9. Methods of ultimate disposal; and
10. Back-up techniques of sludge handling and disposal.

5.7.2. Sludge Conditioning

5.7.2.1. Chemical Conditioning

5.7.2.1.1 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 5.7.

Table 5.7
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 – 50	1.5 – 2.5
R(P + TF)	30 – 60	0 – 150	2 – 5
R(P + AS)	40 – 80	0 – 150	3 – 7.5
AS	60 – 100	50 – 1500	4 – 12.5
DP	20 – 30	30 – 80	1.5 – 4
D(P + TF)	40 – 80	50 – 150	3 – 7.5
D(P + AS)	60 - 100	50 -150	3 - 10

R = Raw P = Primary TF = Trickling Filter AS = Activated Sludge D = Digested

5.7.2.1.2. Laboratory Testing

The selection of the most suitable chemical(s) and the actual dosage requirements for sludge conditioning shall be determined by full-scale testing.

Laboratory testing should, however, 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.

5.7.2.1.3. Conditioning Chemicals

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, although with 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.

5.7.2.1.3.1. Iron or Aluminum Salts

Most raw sludge can be filtered with ferric salts alone, although digested sludge will require an addition of lime with the ferric salt. The lime:ferric chloride ratio is typically 3:1 to 4:1 for best results. If metallic salts are used without lime, the resulting low pH sludge will be highly corrosive to carbon steel and shall require materials such as plastic, stainless steel, or rubber for proper handling.

5.7.2.1.3.2. Lime

Hydrated limes, both the high calcium and dolomitic types, can be used for sludge conditioning in conjunction with metal salts or alone.

5.7.2.1.3.3. 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.

5.7.2.1.3.4. Chemical Feed System

The chemical feed system shall 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 shall be provided so as to disperse the conditioner throughout the sludge. The chemical feed rates should allow for at least a 10:1 range of chemical flow to the dewatering unit.

5.7.2.2. Heat Conditioning

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. Heat conditioning 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 water-solids separation characteristics.

5.7.2.2.1. Operating Temperatures and Pressures

Typical operating temperatures range from150 to 260 °C. Operating pressures range from 1100 to 2800 kPa. Typical sludge detention times vary between 15 and 60 minutes.

5.7.2.2.2. 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 aeration tanks of the sewage treatment plant if supernatant is returned to the aeration system. 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 aeration tanks and allowances must be made in the treatment plant design to accommodate this loading increase.

5.7.2.2.3. Design Considerations

5.7.2.2.3.1. Materials

Heat conditioning results in the production of extremely corrosive liquids requiring the use of corrosion-resistant materials for the liquid handling.

5.7.2.2.3.2. Sludge Grinding

Sludge grinders shall be provided to macerate the sludge to a particle size less than 6 mm to prevent fouling of the heat exchangers.

5.7.2.2.3.3. Feed Pumps

Feed pumps shall be capable of discharging sludge at pressures of 1400 to 2800 kPa and must be resistant to abrasion.

5.7.2.2.3.4. Heat Exchangers

The efficiency of the heat exchangers is dependent on the transfer coefficients and the temperature differences of the incoming and outgoing sludge.

5.7.2.2.3.5. Reaction Vessel

The reaction vessel shall be of sufficient volume to provide for a sludge detention time of 15 to 60 minutes. The detention time depends on the sludge characteristics, temperature and the level of hydrolysis required.

5.7.2.2.3.6. Hot Water Re-circulation Pump

The hot water re-circulation pump shall be capable of handling hot water at a temperature of 25 to 60 ˚C.

5.7.2.2.3.7. 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 oxidating unit should be used.

5.7.2.2.3.8. Solvent Cleaning
Scale formation in the heat exchangers, pipes and reaction vessel require acid washing equipment to be provided.

5.7.2.2.3.9. Piping
All the high pressure piping for the sludge heat conditioning system shall be tested at a pressure of 3500 kPa. Low-pressure piping shall be tested at 1.5 times the working pressure, or 1400 kPa, whichever is greater.

5.7.2.2.3.10. 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}d for primary sludge and 145 kg/m^{2}d for biological sludges. The underflow will range from 10 to 15 % TS.

5.7.2.2.4. 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.

5.7.2.3. 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 or vacuum filters. 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.

5.7.3. Sludge Thickening

5.7.3.1. 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, dissolved-air flotation, centrifuge and others) should consider the type and concentration of sludge, the sludge stabilization processes, 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.

5.7.3.2. 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.

5.7.3.3. Thickener Location

Sludge thickening can be employed in the following locations in a sewage treatment plant:

1. Prior to digestion for raw primary, excess activated sludge or mixed sludge;
2. Prior to dewatering facilities;
3. Following digestion for sludge or supernatant; or
4. Following dewatering facilities for concentration of filtrate, decant, centrate, etc.

Where thickeners are to be housed, adequate ventilation shall be provided.

5.7.3.4. Thickening Methods and 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 5.8. 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 Sludge Volume Indices (SVI's) and benefited by low SVI's in the influent activated sludge. The ranges of thickened sludge concentrations given in Table 5.8 assume an SVI of approximately 100.

Table 5.8
Sludge Thickening Methods and 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
Dissolved Air Flotation	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

5.7.3.5. Sludge Pretreatment

Wherever thickening devices are being installed, special consideration must be given to the need for sludge pretreatment 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 pretreatment.

5.7.3.6. Gravity Thickening

5.7.3.6.1 Process Application

Gravity thickening is principally used for primary sludge, and mixtures of primary and waste activated sludges, with little use for waste activated sludges alone. Due to the better performance of other methods for waste activated sludges, gravity thickening has limited application for such sludges.

5.7.3.6.2. Design Criteria

5.7.3.6.2.1 Tank Shape

The gravity thickener shall be circular in shape.

5.7.3.6.2.2. Tank Dimensions

Typical maximum tank diameters should range between 21 and 24 m. Sidewater depth shall be between 3 and 3.7 m.

5.7.3.6.2.3. Floor Slope

The acceptable range for gravity sludge thickener floor slopes is 2:12 to 3:12.

5.7.3.6.2.4. Solids Loading

The type of sludge shall govern the design value for solids loading to the gravity thickener. Table 5.9 outlines recommended solids loading values.

Table 5.9
Solids Loading on Gravity Thickeners for Various Sludge Types

Type of Sludge	Solids Load (kg/m^2d) 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 waste activated sludge shall be based on a weighted average of the above loading rates. Use of metal salts for phosphorus removal may affect the solids loading rates.

5.7.3.6.2.5. 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 sewage effluent or secondary effluent may be utilized to dilute sludge before thickening.

5.7.3.6.2.6. Hydraulic Overflow Rate

The hydraulic overflow rate shall 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^2s
Secondary Sludge 0.22-0.34 L/m^2s
Mixture 0.25-0.36 L/m^2s

5.7.3.6.2.7. 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 2 days.

Recommended SVR values, in days, are as follows:
1. Warmer months – 0.3 to 1; and
2. Colder months – 0.5 to 2.

5.7.3.6.2.8. Hydraulic Retention Time

A minimum of 6-hour detention of liquid is required. For maximum compaction of the sludge blanket, 24 hours is the recommended time required. During peak conditions, the retention time may have to be shortened to keep the sludge blanket depth below the overflow weirs, thus, preventing excessive solids carry-over.

5.7.3.6.2.9. 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.

5.7.3.6.2.10. 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).

5.7.3.6.2.11. Mechanical Rake

The mechanical rake should have a tip speed of 50 to 100 mm/s. The rake shall be equipped with hinged-lift mechanisms when handling heavy sludges such as lime treated primary sludge. The use of a surface skimmer is recommended.

5.7.3.6.2.12. Overflow Handling

The normal quality of thickener overflow (also known as thickener overhead or supernatant) is about the same as raw sewage 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 shall not be discharged into the secondary settling tank, disinfection tank, sewer outfall, or receiving water.

5.7.3.7. Air Flotation

5.7.3.7.1. Applicability

Unlike heavy sludges, such as primary and mixtures of primary and excess activated sludges, which are generally most effectively thickened in gravity thickeners, light excess activated sludges can be successfully thickened by flotation. 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 excess activated sludges 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.

5.7.3.7.2. 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.

5.7.3.7.3. Design Parameters

The following design parameters are given only as a guide to indicate the normal range of values experienced in full-scale operations.

5.7.3.7.3.1. 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.

5.7.3.7.3.2. Air to Solids Weight Ratio

Typical air to solids weight ratios shall be between 0.02 and 0.05.

5.7.3.7.3.3. Feed Concentration

Feed concentration of activated sludge (including recycle) to the flotation compartment should not exceed 5000 mg/L.

5.7.3.7.3.4. Hydraulic Feed Rate

Where the hydraulic feed rate includes influent plus recycle, the flotation units shall be designed hydraulically to operate in the range of 0.3 to 1.5 L/m^2s. A maximum hydraulic loading rate of 0.5 L/m^2s shall be adhered to when no coagulant aids are used to improve flotation. The feed rate should be continuous rather than on/off.

5.7.3.7.3.5. 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^2d. With the proper addition of flocculating chemicals, the solids loading rate may be increased to 240 kg/m^2d. These loading rates will generally produce a thickened sludge of 3 to 5 % total solids.

5.7.3.7.3.6. Chemical Conditioning

Chemicals used, as coagulant aids shall be fed directly to the mixing zone of the feed sludge and recycle flow.

5.7.3.7.3.7. 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.

5.7.3.7.4. Thickened Sludge Withdrawal

The surface skimmer shall 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. In selecting pumps, the maximum possible sludge concentrations should be taken into consideration.

5.7.3.7.5. Bottom Sludge

A bottom collector to move draw off settled sludge into a hopper must be provided. Draw off from the hopper may be by gravity or pumps.

5.7.3.8. Centrifugation

5.7.3.8.1. Types of Centrifuges

Three types of centrifuges may be utilized for sludge thickening. These include the solid bowl conveyor, disc-nozzle and basket centrifuges.

5.7.3.8.2. Applicability

To date, there has only 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 excess activated sludges. In the way of general comments, the following are given:

1. Centrifugal thickening operations can have substantial maintenance and operating costs;
2. Where space limitations, or sludge characteristics make other methods unsuitable, or where high-capacity mobile units are needed, centrifuges have been used; and
3. Thickening capacity, thickened sludge concentration and solids capture of a centrifuge are greatly dependent on the SVI of the sludge.

5.7.3.8.3. Solids Recovery

The most suitable operating range is generally 85 – 95% solids recovery.

5.7.3.8.4. Polymer Feed Range

A polymer feed range of 0 to 4.0 g/kg of dry solids is generally acceptable.

5.7.4. Anaerobic Sludge Digestion

5.7.4.1. 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. The following is a list of sludges, which are suitable for anaerobic digestion:

1. Primary and lime;
2. Primary and ferric chloride;
3. Primary and alum;
4. Primary and trickling filter;
5. Primary, trickling filter, and alum;
6. Primary and waste activated;
7. Primary, waste activated, and lime;
8. Primary, waste activated, and alum;
9. Primary, waste activated, and ferric chloride; and
10. Primary, waste activated, and sodium aluminate.

5.7.4.1.1. Advantages

The advantages offered by anaerobic digestion include:

1. 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 or produce electricity, or as engine fuel;

2. The quantity of total solids 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 total solids may be destroyed and 40 to 60% of the volatile solids may be destroyed;

3. The 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;

4. Pathogens are destroyed to a high degree during the process. Thermophilic digestion enhances the degree of pathogen destruction; and

5. Most organic substances found in municipal sludge are readily digestible except lignins, tannins, rubber, and plastics.

5.7.4.1.2. Disadvantages

The disadvantages associated with anaerobic digestion include:

1. The digester is easily upset by unusual conditions and erratic or high loadings and very slow to recover;

2. Operators must follow proper operating procedures;

3. Heating and mixing equipment are required for satisfactory performance;

4. Large reactors are required because of the slow growth of methanogens and required solid retention times (SRT's) of 15 to 20 days for a high-rate system. Thus capital cost is high.

5. The resultant supernatant side stream is a strong waste stream that greatly adds to the loading of the wastewater plant. It contains high concentrations of BOD, COD, suspended solids and ammonia nitrogen;

6. Cleaning operations are difficult because of the closed vessel. Internal heating and mixing equipment can become major problems as a result of corrosion and wear in harsh inaccessible environments.

7. A sludge poor in dewatering characteristics is produced;

8. The possibility of explosion as a result of inadequate operation and maintenance, leaks, or operator carelessness exists; and

9. Gas line condensation or clogging can cause major maintenance problems.

5.7.4.2. Digestion Tanks and Number of Stages

With anaerobic sludge digestion facilities, the need for multiple units can often be avoided by providing two-stage digestion along with sufficient flexibility in sludge pumpage and mixing so that one stage can be serviced while the other stage receives the raw sludge pumpage. Single stage digesters will generally not be satisfactory due to the usual need for sludge storage, and effective supernating. 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.

5.7.4.3. Access Manholes

At least two, 1-meter 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 shall be provided. The opening should be large enough to permit the use of mechanical equipment to remove grit and sand. This manhole shall be located near the bottom of the sidewall. All manholes shall be provided with gas-tight and watertight covers.

5.7.4.4. 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 shall be provided for workers involved in cleaning the digesters.

Necessary safety facilities shall be included where sludge gas is produced. All tank covers shall be provided with pressure and vacuum relief valves and flame traps together with automatic safety shut-off valves. Water seal equipment shall not be installed.

5.7.4.5. 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.

5.7.4.6. Typical Sludge Qualities and Generation Rates for Different Unit Processes

When reliable data are not available, the sludge generation rates and characteristics given in Table 5.10 may be used.

Table 5.10
Typical Sludge Qualities and Generation Rates

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	1.5 – 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	160	62
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 Anaerobic Digestion						
Undigested (no P removal)	15.5	0.4–2.6	1.1	70	170	77
Undigested (with P removal)	19.1	0.4-2.6	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 (P removal)	13.3	1.4-1.9	0.9	60	120	55
Sludge Holding Tank (no P removal)	4.0	1.4-5.0	2.0	70	80	36
Sludge Holding Tank (P removal)	5.5	0.4-4.5	2.0	60	110	50

Note:
1. L/m^3 denotes litres of liquid sludge per cubic metre of treated sewage
2. g/m^3 denotes grams of dry solids per cubic metre of treated sewage
3. the above values are based on typical raw sewage with total BOD = 570 mg/L, soluble BOD = 50, SS = 200 mg/L, P = 7 mg/L, NH_3 = 20 mg/L

5.7.4.7. Solids Retention Time

The minimum solids retention time for a low rate digester shall be 30 days. The minimum solids retention time of a high rate digester shall be 15 days.

5.7.4.8. Design of Tank Elements

5.7.4.8.1. 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 heat loss viewpoint. However, structural requirements and scum control aspects also govern the optimum depth-diameter ratio.

5.7.4.8.2. Floor Slope

To facilitate draining, cleaning and maintenance, the following features are desirable:

1. The tank bottom should slope to drain toward the withdrawal pipe;

2. For tanks equipped with mechanisms for withdrawal of sludge, a bottom slope not less than 1:12 (vertical:horizontal) is recommended; and

3. Where the sludge is to be removed by gravity alone, 1:4 slope is recommended.

5.7.4.8.3. Depth and 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.

5.7.4.8.4. Scum Control

Scum accumulation can be controlled by including any of the following provisions in the equipment design:

1. Floating covers keep the scum layer submerged and thus moist and more likely to be broken up;

2. Discharging re-circulated sludge on the scum mat serves the same purpose as (1);

3. Re-circulating sludge gas under pressure through the tank liquors and scum;

4. Mechanically destroying the scum by employing rotating arms or a propeller in a draft tube;

5. A large depth-area ratio; or

6. A concentrated sludge feed to the digester.

Items (5) and (6) would release large volumes of gas per unit area, keep the scum in motion and mix the solids in the digester.

5.7.4.8.5. Grit and 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 1:4.

5.7.4.8.6. Alkalinity and 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 buffer capacity of a freshwater system. Higher alkalinity values indicate a greater capacity for resisting pH changes. The alkalinity shall be measured as bicarbonate alkalinity. Values for alkalinity in anaerobic digesters range from 1500 to 5000 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 shall 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.

5.7.4.8.7. 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 shall assure the homogeneity of the digester contents, and prevent stratification.

Gas mixing methods are preferred. Gas mixing may be accomplished in any one of the following manners:

1. Short mixing tubes;
2. One or more deep-draft tubes;
3. Diffusers at the digester floor; or
4. Gas discharge below scum level.

5.7.4.8.8. Sludge Inlets, Outlets, Re-circulation, and High Level Overflow

5.7.4.8.8.1. Multiple Inlets and Draw-Offs

Multiple sludge inlets and draw-offs and, where used, multiple re-circulation suction and discharge points to facilitate flexible operation and effective mixing of the digester contents, shall be provided unless adequate mixing facilities are provided within the digester.

5.7.4.8.8.2. Inlet Configurations

One inlet should discharge above the liquid level and be located at approximately the centre of the tank to assist in scum break-up. The second inlet should be opposite to the suction line at approximately the 0.7 diameter point across the digester.

5.7.4.8.8.3. Inlet Discharge Location

Raw sludge inlet discharge points should be so located as to minimize short-circuiting to the digester sludge or supernatant draw-offs.

5.7.4.8.8.4. 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 re-circulation piping, to increase operational flexibility in mixing the tank contents.

5.7.4.8.8.5. Emergency Overflow
An unvalved vented overflow shall be provided to prevent damage to the digestion tank and cover in case of accidental overfilling. This emergency overflow shall be piped to an appropriate point and at an appropriate rate in the treatment process or sidestream treatment facilities to minimize the impact on process units.

5.7.4.8.9. Primary Tank Capacity
The primary digestion tank capacity should be determined by rational calculations based upon such factors as volume of sludge added, its 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 shall 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 Sections 5.7.4.8.9.1 and 5.7.4.8.9.2 will be required. Such requirements assume that a raw sludge is derived from ordinary domestic wastewater, that a digestion temperature is to be maintained in the range of 32 °C 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.

5.7.4.8.9.1. High Rate Digester
The primary high rate digester shall 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 1.6 kg of volatile solids per cubic meter 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.

5.7.4.8.9.2. 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 cubic meter of volume per day in the active digestion unit. This loading may be modified upward or downward depending upon the degree of mixing provided.

5.7.4.8.10. 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 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 2:1 to 4:1 are recommended).

5.7.4.8.11. 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 shall be on the secondary digester. Insulated pressure and vacuum relief valves and flame traps shall 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 can also be successfully employed, such as concrete and fibreglass.

5.7.4.8.12. 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 shall 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).

5.7.4.8.13. Overflows

Each digester should be equipped with an emergency overflow system.

5.7.4.9. Gas Collection, Piping and Appurtenances

All portions of the gas system including the space above the tank liquor, storage facilities and piping shall 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 shall be adequately ventilated. All gas collection equipment, piping and appurtenances shall comply with the Canadian Gas Association Standard B105-M93.

5.7.4.9.1. Safety Equipment

All necessary safety facilities shall be included where gas is produced. Pressure and vacuum relief valves and flame traps together with automatic safety shut-off valves, are essential. Water seal equipment shall 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 shutdown or pilot light failure, and before it can be ignited. This will provide certainty that no explosive mixture exists within the unit.

5.7.4.9.2. Gas Piping and Condensate

The main gas collector line from the digestion tanks shall be at least 64 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. 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 estimate 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 shall be limited to not more than 3.7 m/s.

Gas piping shall slope to condensation traps at low points. 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.

5.7.4.9.3. 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 shall be provided with suitable flame traps.

5.7.4.9.4. Electrical Systems

Electrical fixtures and controls, in places enclosing anaerobic digestion appurtenances, where hazardous gases are normally contained in the tanks and piping, shall comply with the Canadian Electrical Code, Part 1 and the applicable provincial power standards. Digester galleries should be isolated from normal operating areas.

5.7.4.9.5. Waste Gas

Waste gas burners shall be readily accessible and should be located at least 15 m away from any plant structure if placed at ground level, or may be located on the roof of the control building if sufficiently removed from the tank. In remote locations it may be permissible to discharge the gas to the atmosphere through a return-bend screened vent terminating at least 3 m above the walking surface, provided the assembly incorporates a flame trap. Waste gas burners shall be of sufficient height and so located to prevent injury to personnel due to wind or downdraft conditions.

All waste gas burners shall 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 always required.

5.7.4.9.6. Ventilation

Any underground enclosures connecting with digestion tanks or containing sludge or gas piping or equipment shall be provided with forced ventilation in accordance with Section 5.3.10. Tightly fitting self-closing doors should be provided at connecting passageways and tunnels to minimize the spread of gas.

5.7.4.9.7. Meter

A gas meter with bypass shall be provided, to meter total gas production for each active digestion unit. A single gas meter with proper interconnected gas piping may measure total gas production for two-stage digestion systems operated in series. Where multiple primary digestion units are utilized with a single secondary digestion unit, a gas meter shall 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 shall be properly valved with gas tight gate 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 or vortex type. Positive displacement meters should not be utilized. The meter must be specifically designed for contact with corrosive and dirty gases.

5.7.4.10. Digestion Tank Heating

5.7.4.10.1. Heating Capacity

Sufficient heating capacity shall 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.

5.7.4.10.2. Insulation

Wherever possible, digestion tanks should be constructed above ground-water level and should be suitably insulated to minimize heat loss.

5.7.4.10.3. 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.

5.7.4.10.3.1. External Heating

Piping shall be designed to provide for the preheating of feed sludge before introduction to the digesters. Provisions shall be made in the layout of the piping and valving to facilitate heat exchanger tube removal and cleaning of these lines. Heat exchanger sludge piping shall 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.

5.7.4.10.3.2. Other Heating Methods

1. 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.

2. 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, operation, and maintenance characteristics will be required.

5.7.4.10.4. Hot Water Internal Heating Controls

5.7.4.10.4.1. Mixing Valves

A suitable automatic mixing valve shall 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.

5.7.4.10.4.2. Boiler Controls

The boiler should be provided with suitable automatic controls to maintain the boiler temperature at approximately 82 ºC to minimize corrosion and to shut off the main gas supply in the event of pilot burner or electrical failure, low boiler water level, excessive temperature, or low gas pressure.

5.7.4.10.4.3. Boiler Water Pumps

Boiler water pumps shall be sealed and sized to meet the operating conditions of temperature, operating head, and flow rate. Duplicate units shall be provided.

5.7.4.10.4.4. Thermometers

Thermometers should be provided to show inlet and outlet temperatures of sludge, hot water feed, hot water return and boiler water.

5.7.4.10.4.5. Water Supply

The chemical quality should be checked for suitability for this use.

5.7.4.10.5. 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.

5.7.4.11. Supernatant Withdrawal

5.7.4.11.1. 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.

5.7.4.11.2. Withdrawal Arrangement

5.7.4.11.2.1. Withdrawal Levels

Piping should be arranged so that withdrawal can be made from three or more levels in the tank. A positive unvalved vented overflow shall be provided. 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.

5.7.4.11.2.2. Supernatant Selector

A fixed screen supernatant selector or similar type device shall be limited for use in an unmixed secondary digestion unit.

If a supernatant selector is provided, provisions shall 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 backwash facilities shall be provided.

5.7.4.11.2.3. 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.

5.7.4.11.3. 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.

5.7.4.11.4. 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.

5.7.4.12. 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 shall govern the design:

1. To avoid clogging, sludge sampling pipes should be at least 75 mm in diameter;

2. Provision should be made for the connection of a water source of adequate pressure to these pipes for back flushing when the need arises; and

3. There shall be at least three sampling pipes each separately valved for the primary digesters and four for the secondary digesters.

5.7.5. 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.

5.7.5.1. Applicability

Aerobic digestion is considered suitable for secondary sludge or a combination of primary and secondary sludge. Table 5.11 presents the advantages and disadvantages in the use of aerobic sludge digestion.

Table 5.11
Advantages and Disadvantages of Aerobic Sludge Digestion

Advantages	Disadvantages
Low initial cost, especially for small plants	High energy costs
Supernantant less objectionable than anaerobic	Generally lower VSS destruction than anaerobic
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

5.7.5.2. 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.

5.7.5.3. Multiple Units

Multiple digestion units capable of independent operation are desirable and shall be provided in all plants where the design average flow exceeds 455 m^3/d. All plants not having multiple units shall provide alternate sludge handling and disposal methods.

5.7.5.4. Pretreatment

Thickening of sludge is recommended prior to aerobic digestion.

5.7.5.5. Design Considerations

Factors, which should be considered when designing aerobic digesters, include:
1. Type of sludge to be digested;
2. Ultimate method of disposal;
3. Required winter storage;
4. Digester pH;
5. Sludge temperature; and
6. Raw sludge qualities.

5.7.5.6. Solids Retention Time

Where land disposal of digested sludge is practised, a minimum solids retention time of 45 days is required. If local conditions require a more stable sludge, a sludge age of 90 days shall be necessary. To produce a completely stable sludge, a sludge age in excess of 120 days is required.

5.7.5.7. Hydraulic Retention Time

The minimum required hydraulic retention time (HRT) for aerobic digesters provided with pre-thickening facilities are as listed in Table 5.12:

Table 5.12
Minimum HRT

Minimum HRT (days)	Type of Sludge
25	Waste Activated Sludge Only
25	Trickling Filter Sludge Only
30	Primary Plus Secondary Sludge

The more critical of the two guidelines, solids retention time and hydraulic retention time, shall govern the design.

5.7.5.8. Tank Design

5.7.5.8.1. Tank Capacity

The determination of tank capacities shall be based on rational calculations, including factors such as quantity of sludge produced, sludge characteristics, time of aeration and sludge temperature.

Calculations shall be submitted to justify the basis of design. When such calculations are not based on the above factors, the minimum combined digestion tank capacity shall be based on the following:
1. Volatile solids loading shall not exceed 1.60 kg/m^3d in the digestion units. Lower loading rates may be necessary depending on temperature, type of sludge and other factors.

2. If a total of 45 days sludge age is all that is provided, it is suggested that ⅔ of the total digester volume be in the first tank and ⅓ 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.

5.7.5.8.2. Air and Mixing Requirements

Aerobic sludge digestion tanks shall be designed for effective mixing by satisfactory aeration equipment. Sufficient air shall be provided to keep the solids in suspension and maintain dissolved oxygen from 1 to 2 mg/L. A minimum mixing and air requirement of 0.85 L/s/m^3 of tank volume shall 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. If mechanical aerators are utilized, at least two turbine aerators per tank shall be provided. Use of mechanical equipment is discouraged where freezing temperatures are normally expected.

Air supply to each tank should be separately valved to allow aeration shutdown in either tank.

5.7.5.8.3. 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 shall be between 3.5 to 4.5 m; tanks and piping should be designed to permit sludge addition, sludge withdrawal, and supernatant decanting 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 1:12 to 3:12 should be provided.

5.7.5.8.4. Supernatant Separation and Scum and Grease Removal

Facilities shall be provided for effective separation or decanting of 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 shall be designed to prevent recycle of scum and grease back to plant process units. Provision should be made to withdraw supernatant from multiple levels of the supernatant withdrawal zone.

Facilities shall 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.

5.7.5.9. High Level Emergency Overflow

An unvalved high level overflow and any necessary piping shall 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 shall include waste sludge rate and duration during the period the plant is unattended, potential effect on plant process units, discharge location of the emergency overflow, and potential discharge or suspended solids in the plant effluent.

5.7.5.10. Mixing Tanks and Equipment

Mixing tanks may be designed to operate as either a batch or continuous flow process. A minimum of two tanks shall be provided of adequate size to provide a minimum 2 hours contact time in each tank. The following items shall be considered in determining the number and size of tanks:

1. Peak sludge flow rates;
2. Storage between batches;
3. Dewatering or thickening performed in tanks;
4. Repeating sludge treatment due to pH decay of stored sludge;
5. Sludge thickening prior to sludge treatment; and
6. Type of mixing device used and associated maintenance or repair requirements.

Mixing equipment shall 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 0.85 L/m^3s of mixing tank volume shall 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 shall be designed to minimize fouling with debris in the sludge and consideration shall be made to provide continuity of service during freezing weather conditions.

5.7.5.11. Chemical Feed and Storage Equipment

Alkaline material is caustic in nature and can cause eye and tissue injury. Equipment for handling or storing alkaline material shall 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 shall be accessible for cleaning.

5.7.5.11.1. Feed and Slaking Equipment

The design of the feeding equipment shall 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 shall be sized to handle a minimum of 150% of the peak sludge flow rate including sludge that may need to be retreated due to pH decay. Duplicate units shall be provided.

5.7.5.11.2. 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 shall contain a mechanical agitation mechanism. Storage facilities shall be sized to provide a minimum of a 30-day supply.

5.7.5.12. Sludge Storage

The design shall incorporate considerations for the storage of high pH stabilized sludge, as per the following sub-sections.

5.7.5.12.1. Liquid Sludge

Liquid high pH stabilized sludge shall not be stored in a lagoon. Said sludge shall be stored in a tank or vessel equipped with rapid sludge withdrawal mechanisms for sludge disposal or re-treatment. Provisions shall be made for adding alkaline material in the storage tank. Mixing equipment in accordance with Section 5.7.5.10 shall also be provided in all storage tanks.

5.7.5.12.2. Dewatered Sludge

On-site storage of dewatered high pH stabilized sludge should be limited to 30 days. Provisions for rapid re-treatment or disposal of dewatered sludge stored on-site shall also be made in case of sludge pH decay.

5.7.5.12.3. Off-Site Storage

There shall be no off-site storage of high pH stabilized sludge unless specifically permitted the DOEC.

5.7.5.13. Sludge Disposal

Immediate sludge disposal methods and options are recommended to be utilized in order 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. If the land application disposal option is utilized for high pH stabilized sludge, said sludge must be incorporated into the soil during the same day of delivery to the site.

5.7.6. Sludge Dewatering

Sludge dewatering will often be required at sewage treatment plants prior to ultimate disposal of sludges. 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 sewage treatment plant. 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 waste activated sludge, 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).

As with 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. Also, adequate ventilation equipment will be required in buildings housing dewatering equipment. 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 sewage 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 shall be developed and provided to the regulatory agencies for approval on a case-by case basis.

5.7.6.1. Dewatering Process Compatibility with Subsequent Treatment or Disposal Techniques

Table 5.13 outlines the relationship of dewatering to other sludge treatment processes.

Table 5.13
Relationship of Dewatering to Other Sludge Treatment Processes
For Typical Municipal Sludge

Method	**Pretreatment Normally Provided**		**Normal Use of Dewatering Cake**			
	Thickening	**Conditioning**	**Landfill**	**Land Spread**	**Heat Drying**	**Incineration**
Rotary Vacuum Filter	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

5.7.6.2. Sludge Drying Beds

5.7.6.2.1. Pretreatment

Sludge shall be pretreated before being air-dried by either one of the following methods:

1. Anaerobic digesters;
2. Aerobic digesters with provision to thicken;
3. 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; or
4. Well designed and maintained oxidation ditches with sludge age longer than about 20 days (preferably after thickening).

5.7.6.2.2. 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 one-third or less, of the unconditioned drying time, provision should be made for the addition of conditioning chemicals, usually polymers.

5.7.6.2.3. Design Criteria

5.7.6.2.3.1. Factors Influencing Design

The design and operation of sludge drying beds depend on the following factors:

1. Climate in the area;
2. Sludge characteristics;
3. Pre-treatment (such as conditioning, thickening, etc.); and
4. Sub-soil permeability.

5.7.6.2.3.2. Bed Area

Consideration should be given to the following when calculating the bed area:

1. The volume of wet sludge produced by existing and proposed processes;

2. Depth of wet sludge drawn to the drying beds. For design calculation purposes a maximum depth of 200 mm shall 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 and type of digestion utilized;

3. Total digester volume and other wet sludge storage facilities;

4. Degree of sludge thickening provided after digestion;

5. The maximum drawing depth of sludge, which can be, removed from the digester or other sludge storage facilities without causing process or structural problems;

6. The time required on the bed to produce a removable cake. Adequate provision shall 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; and

7. 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 DOEC supporting any design based on the above methods. In the absence of such calculations the minimum sludge drying bed shall be based on the criteria presented in Table 5.14.

Table 5.14
Sludge Drying Bed Areas

Type of Wastewater Treatment	Area (m^3/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.

5.7.6.2.3.3. Percolation Type Beds

1. **Pond Bottom** - The bottom of the cell should be of impervious material such as clay or asphalt.

2. **Underdrains** - Underdrains should 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 slope of 1.0 %, or more. Underdrains should discharge back to the secondary treatment section of the sewage treatment plant. Various pipe materials may be selected provided the material is of suitable strength and corrosion resistant.

3. **Gravel** - The lower course of gravel around the underdrains should be properly graded and should be 300 mm in depth, extending at least 150 mm above the top of the underdrains. It is desirable to place this in two or more layers. The top layer, of at least 75 mm in depth, should consist of gravel 3 mm to 6 mm in size. The gravel should be graded from 25 mm on the bottom to 3 mm on the top.

4. **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 uniformity co-efficient of less than 5.0. The finished sand surface should be level.

5. **Additional Dewatering Provisions** - Consideration shall 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.

5.7.6.2.3.4. Impervious Type Beds

Paved drying beds should be designed with consideration for space requirements to operate mechanical equipment for removing the dried sludge.

5.7.6.2.3.5. Location

Depending on prevailing wind directions, a minimum distance of 100 to 150 m shall be kept from open sludge drying beds and dwellings. However, the minimum maybe reduced to 60 m to 80 m for enclosed beds. The selected location for open beds shall be at least 30 m from public roads and 25 m for enclosed beds. The plant owner may be required to spray deodorants and odour masking chemicals whenever there are complaints from the population in the neighbourhood.

5.7.6.2.3.6. 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 March).

5.7.6.2.3.7. Dimensions

The bed size generally should be 4.5 to 7.5 m wide with the length selected to satisfy desired bed loading volume.

5.7.6.2.3.8. Depth of Sludge

The sludge dosing depth shall generally be 200 to 300 mm for warm weather operating modes; for winter freeze drying depths of 1 to 3 m can be used depending upon the number of degree days in winter.

5.7.6.2.3.9. Number of Beds

Three beds are desirable for increased flexibility of operation. Not less than two beds shall be provided.

5.7.6.2.3.10. Walls

Walls should be watertight and extend 400 to 500 mm above and at least 150 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.

5.7.6.2.3.11. 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.

5.7.6.2.3.12. Sludge Removal

Each bed shall be constructed so as to be readily and completely accessible to mechanical cleaning equipment. Concrete runways spaced to accommodate mechanical equipment shall 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 shall 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 3 or 4 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 10 with covered beds. These values are tentative and subject to revision after field observations.

5.7.6.2.3.13. Covered Beds
Consideration should be given to the design and use of covered sludge drying beds.

5.7.6.3. Sludge Lagoons
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 sludge, limed sludge, or sludge 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; precipitation and low temperatures 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 shall be made for other sludge dewatering facilities or sludge disposal in the event of upset or failure of the sludge digestion process.

5.7.6.3.1. Design Considerations
The design and location of sludge lagoons must take into consideration many factors, including the following:
1. Possible nuisances – odours, appearances, mosquitoes;
2. Design – number, size, shape and depth;
3. Loading factors – solids concentration of digested sludge, loading rates;
4. Soil conditions – permeability of soil, need for liner, stability of berm slopes, etc.;
5. Groundwater conditions – elevation of maximum groundwater, level, direction of groundwater movement, location of wells in the area;
6. Sludge and supernatant removal – volumes, concentrations, methods of removal, method of supernatant treatment and final sludge disposal; and
7. Climatic effects – evaporation, rainfall, freezing, snowfall, temperature, solar radiation.

5.7.6.3.2. Pre-Treatment
Pre-treatment requirements for sludge lagoons are the same as those for sludge drying beds.

*5.7.6.3.3. **Soil and Groundwater Conditions***

The soil must be reasonably porous and the bottom of the lagoons must be at least 1.2 m above the maximum ground water table. Surrounding areas shall be graded to prevent surface water entering the lagoon. In some critical instances, the DOEC may require a lagoon to be lined with plastic or rubber material.

*5.7.6.3.4. **Depth***

Lagoons should be at least 1 m in depth while maintaining a minimum of 0.6 m of freeboard.

*5.7.6.3.5. **Seal***

Adequate provisions shall be made to seal the sludge lagoon bottom and embankments in accordance with the requirements of Section 5.9.6.2 to prevent leaching into adjacent soils or ground water.

*5.7.6.3.6. **Area***

The area required will depend on local climatic conditions. Not less than two lagoons should be provided.

*5.7.6.3.7. **Location***

Consideration shall be given to prevent pollution of ground and surface water. Adequate solation shall be provided to avoid nuisance production.

*5.7.6.3.8. **Cycle Time and 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 %.

5.7.6.4. Mechanical Dewatering Facilities

Provisions shall 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 vacuum filters, centrifuges, filter presses or belt filters, a detailed description of the process and design data shall accompany the plans. Unless standby facilities are available, adequate storage facilities shall be provided. The storage capacity should be sufficient to handle at least 4 days of sludge production volume.

*5.7.6.4.1. **Performance of Mechanical Dewatering Methods***

Table 5.15 outlines the solids capture, solids concentrations normally achieved and energy requirements for various mechanical dewatering methods.

Table 5.15
Sludge Dewatering Methods and Performance with Various Sludge Types

Dewatering Method	Solids Capture (%)	Solids Concentrations Normally Achieved [(1)]	Median Energy Required (MJ/Dry Tonne) [(2)]
Vacuum Filter	90 – 95	Raw primary + was (10-25%) Digested primary + was (15-20%) Was (8-12%)	1080
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	85 - 95	Raw or Digested primary + was (14-25%) Was (10-15%)	130

1. Including conditioning chemicals, if required.
2. MJ/Dry Tonne – denotes mega joules per dry tonne of sludge throughout.

5.7.6.4.2. 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. There shall be a back up pump and filtrate pump installed for each vacuum filter.

5.7.6.4.3. Ventilation

Adequate facilities shall be provided for ventilation of the dewatering area. The exhaust air should be properly conditioned to avoid odour nuisance.

5.7.6.4.4. 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.

5.7.6.4.5. Drainage and Filtrate Disposal

Drainage from beds or filtrate from dewatering units shall be returned to the sewage treatment process at appropriate points.

5.7.6.4.6. 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 shall accompany the plans.

5.7.6.4.7. Vacuum Filters

Of primary importance with vacuum filters is the solids concentration of sludge fed to the units. With all other operating variables remaining constant, increases in filtration rates vary in direct proportion to feed solids. Sludge thickening prior to vacuum filters is therefore extremely important. Higher concentrations in the sludge feed also result in lower filtrate solids.

Vacuum filtration systems should be designed in accordance with the following parameters:

1. Sludge feed pumps:
   a) Variable capacity;

2. Vacuum pumps:
   a) Generally one per machine with capacity of 10 L/m^{2}s at 65 kPa or more, vacuum;

3. Vacuum receiver:
   a) Generally one per machine;
   b) Maximum air velocity 0.8 to 1.5 m/s;
   c) Air retention time 2-3 minutes;
   d) Filtrate retention time 4-5 minutes;
   e) All lines shall slope downward to the receiver from the vacuum filter;

4. Filtrate pumps:
   a) Generally self-priming centrifugal;
   b) Suction capacity greater than vacuum pump, 65 to 85 kPa vacuum;
   c) With flooded pump suctions;
   d) With check valve on the discharge side to minimize air leakage into the system;
   e) Pumps must be sized for the maximum expected sludge drainage rates (usually produced by polymers);

5. Sludge flocculation tank:
   a) Constructed of corrosion-resistant materials; and
   b) With slow speed variable drive mixer, detention time 2-4 minutes with ferric and lime (with polymers shorter time may be used);

6. Wash water:
   a) Filtered final effluent generally used;

7. Sludge measurement:
   a) Should be provided unless measured elsewhere in plant;

8. Solids loading rate:
   a) 7-14 g/m^{2}s for raw primary;
   b) 2.75-7 g/m^{2}s for raw primary + WAS; 4-7 g/m^{2}s for digested primary + WAS; and
   c) Not considered practical for use with WAS alone.

5.7.6.4.8. Filter Presses

As with vacuum filters, 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:

1. Sludge conditioning tank:
   a) Detention time maximum 20 minutes at peak pumpage rate;

2. Feed Pumps:
   a) Variable capacity to allow pressures to be increased gradually, without underfeeding or overfeeding sludge;
   b) Pumps should be of a type to minimize floc shear;
   c) Pumps must deliver high volume at low head initially and low volume at high head during latter part of cycle; and
   d) Ram or piston pumps, progressing cavity pumps or double diaphragm pumps are generally used;

3. Cake handling:
   a) Filter press must be elevated above cake conveyance system to allow free fall; and
   b) Cake can be discharged directly to trucks, into dumper boxes, or onto conveyors (usually cable cake breakers may be needed);

4. Cycle times:
   a) 1.5 to 6 hours (normally 1.5 to 3 hours); and

5. Operating pressures:
   a) Usually 700 to 1400 kPa, but may be as high as 1750 kPa. The operating pressure shall not exceed 1000 to 1050 kPa, if polymer is applied as the conditioning agent.

5.7.6.4.9. Solid Bowl Centrifuges

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 counter-current or concurrent. Pool depth can be varied by adjustable weirs. 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. Other general design guidelines for solid bowl centrifuges are as follows:

1. Feed pump:
   a) Sludge feed should be continuous;
   b) Pumps should be variable flow type;
   c) One pump should be provided per centrifuge for multiple centrifuge systems; and
   d) Chemical dosage should vary with the pumpage rate;

2. Sludge pre-treatment:
   a) Depending upon the sewage treatment process, grit removal, screening or maceration may be required for the feed sludge stream;

3. Solids Capture:
   a) 85 to 95% is generally desirable;

4. Machine materials:
   a) Generally carbon steel or stainless steel; and
   b) Parts subject to wear should be protected with hard facing materials such as a tungsten carbide material;

5. Machine foundations:
   a) Foundations must be capable of absorbing the vibratory loads;

6. Provision for maintenance:
   a) Sufficient space must be provided around the machine(s) to permit disassembly;
   b) An overhead hoist should be provided;
   c) Hot and cold water supplies will be needed to permit flushing out the machine; and
   d) Drainage facilities will be necessary to handle wash water.

5.7.6.4.10. Belt Filter Presses
Most types of wastewater sludges can be dewatered with belt filter presses and the results achieved are generally superior to those of vacuum filters. 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.

5.7.7. Sludge Pumps and Piping

5.7.7.1. Sludge Pumps

5.7.7.1.1. General Sludge Pumping Requirements
Table 5.16 outlines general sludge pumping requirements for various sludge types.

Table 5.16
Sludge Pumping Requirements for Various Sludge Types

Sludge Source	**Slurry** (% total solids)	**Static Head** (m)	**TDH** (m)	**Abrasive Service**	**Duty**
Pre-treatment - Grit	0.5 – 10	0 – 1.5 (gravity)	1.5 – 3	Yes – High	Heavy
Primary Sedimentation Unthickened Thickened	0.2 – 2 4 – 10	3 – 12 3 – 12	10 – 200 12 – 25	Yes Yes	Medium Heavy
Secondary Sedimentation (for re-circulation)	0.5 – 2	1 – 2	3 – 4.5	No	Light
Secondary Sedimentation (for thickening)	0.5 – 2	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 Re-circulation Underflow	3 – 10 3 – 10	0 – 1.5 0 – 6	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 1 – 6 2 – 15	 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

Note: * Depends on degritting efficiency
** High pressure for heat treatment

5.7.7.1.2. Capacity
Pump capacities should be adequate but not excessive. Provision for varying pump capacity is desirable.

5.7.7.1.3. Duplicate Units
Duplicate units shall be provided where failure of one unit would seriously hamper plant operation.

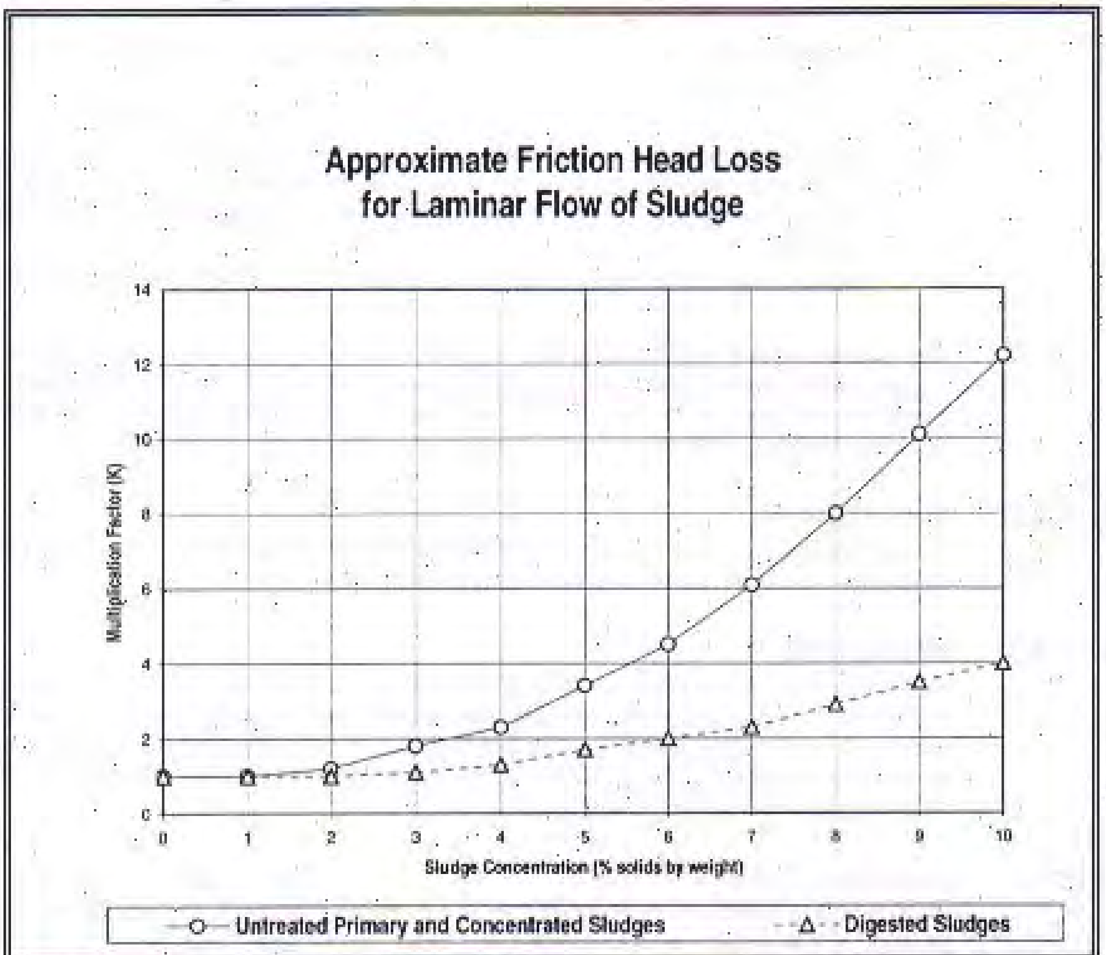
5.7.7.1.4. Type
Plunger pumps, screw feed pumps or other types of pumps with demonstrated solids handling capability should be provided for handling raw sludge. Where centrifugal pumps are used, a parallel positive displacement pump should be provided as an alternate to pump heavy sludge concentrations, such as primary or thickened sludge, that may exceed the pumping head of the centrifugal pump.

5.7.7.1.5. Minimum Head
A minimum positive head of 600 mm shall 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.

5.7.7.1.6. Head Loss

Figure 5.2 shows the multiplication factor to apply to the friction losses for turbulent flow for clean water to calculate the friction losses for untreated primary and concentrated sludges and digested sludge. Use of Figure 5.2 will often provide sufficiently accurate results for design, especially at solids concentrations below 3 %. However, as pipe length, percent total solids and percent volatile solids increase, more elaborate methods may have to be used to calculate the friction losses with sufficient accuracy.

Figure 5.2



NOTES: 1. Multiply loss with clean water by K to estimate friction loss under laminar conditions (see text).

2. The information on this figure has been extracted from EPA 625/1-79-011 "Process Design Manual for Sludge Treatment and Disposal:, September 1979.

5.7.7.1.7. Sampling Facilities

Unless sludge-sampling facilities are otherwise provided, quick closing sampling valves shall 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.

5.7.7.2. Sludge Piping

5.7.7.2.1. Size and 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. With sludge pumpage 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.

5.7.7.2.2. Slope

Gravity piping should be laid on uniform grade and alignment. The slope on gravity discharge piping should not be less than 3 percent. Provisions should be made for draining and flushing discharge lines.

5.7.7.2.3. Supports

Special consideration should be given to the corrosion resistance and continuing stability of supporting systems for piping located inside the digestion tank.

5.8. Biological Treatment

5.8.1. Activated Sludge

5.8.1.1. Applicability

The activated sludge process and its various modifications may be used where sewage 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.

5.8.1.2. Process Selection

The activated sludge process and its several modifications may be employed to accomplish varied degrees of removal of suspended solids and reduction of carbonaceous and/or nitrogenous oxygen demand. Choice of the process most applicable will be influenced by the degree and consistency of treatment required, type of waste to be treated, proposed plant size, anticipated degree of operation and maintenance, and operating and capital costs. All designs should provide for flexibility in operation. Plants over 4500 m^3/d should be designed to facilitate easy conversion to various operation modes, if feasible.

The design must be based on experience at other facilities. Continuity and reliability of treatment equal to that of the continuous flow through modes of the activated sludge process shall be provided. The DOEC shall be contacted for design guidance and criteria where such systems are being considered.

5.8.1.3. 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.

5.8.1.4. Winter Protection

Protection against freezing shall be provided to ensure continuity of operation and performance.

5.8.1.5. Pretreatment

Where primary settling tanks are not used, effective removal or exclusion of grit, debris, excessive oil or grease, and comminution or screening of solids shall be accomplished prior to the activated sludge process.

Where primary settling is used, provision shall be made for discharging raw sewage directly to the aeration tanks to facilitate plant start-up and operation during the initial stages of the plant's design life.

5.8.1.6. Aeration

5.8.1.6.1. Capacities and Permissible Loadings

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 mixed liquor suspended solids 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-dissolved oxygen should be considered when designing for nitrification.

Calculations should be submitted to justify the basis for design of aeration tank capacity. Calculations using values differing substantially from those in the accompanying table should reference actual operational plants. Mixed liquor suspended solids levels greater than 5000 mg/L may be allowed providing adequate data is submitted showing the aeration and clarification system capable of supporting such levels.

When process design calculations are not submitted, the aeration tank capacities and permissible loadings for several adaptations of the processes shown in Table 5.17 shall be used. These values apply to plants receiving diurnal load ratios of design peak hourly BOD_5 to design average BOD_5 ranging from about 2:1 to 4:1. Thus, the utilization of flow equalization facilities to reduce the diurnal design peak hourly BOD_5 organic load may be considered by the DOEC as justification to approve organic loading rates that exceed those specified in Table 5.17.

Table 5.17
Allowable Aeration Tank Capacity and Loading

Type of Process	Percentage of Average Design Flow		
	Minimum (%)	Normal (%)	Maximum (%)
Plug Flow	25	30	100
Complete Mix	25	30	100
Carbonaceous Stage of Separate Sludge	25		75
Nitrification	25	50	75
Step Aeration	50	100	150
Contact Stabilization	50	100	150
Extended Aeration	50	100	150
Oxidation Ditch	50	50	200
High Rate Nitrification Stage of Separate Stage Nitrification	50		200

5.8.1.6.2. Arrangement of Aeration Tanks

1. Dimensions
   The dimensions of each independent mixed liquor aeration tank or return sludge re-aeration tank shall be such as to maintain effective mixing and utilization of air. Ordinarily, liquid depths should not be less than 3 m or more than 9 m except in special design cases.

2. Short Circuiting
   For very small tanks or tanks with special configuration, the shape of the tank and the installation of aeration equipment should provide for positive control of short-circuiting through the tank.

5.8.1.6.2.1. Number of Units

Total aeration tank volume should be divided among two or more units, capable of independent operation, when required by the DOEC to meet applicable effluent limitations and reliability guidelines.

5.8.1.6.2.2. Inlets, Outlets and Conduits

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 reasonably constant liquid level. The effluent weir for a horizontally mixed aeration tank system must be easily adjustable by mechanical means and shall be sized based on the design peak instantaneous flow plus the maximum return sludge flow. The hydraulic properties of the system should permit the design peak instantaneous flow to be carried with any single aeration tank unit out of service.

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.

5.8.1.6.2.3. Freeboard

All aeration tanks should have a freeboard of not less than 450 mm. However, if a mechanical surface aerator is used, the freeboard should not be less than 1 m to protect against windblown spray freezing on walkways, etc.

5.8.1.6.3. Aeration Equipment

Oxygen requirements generally depend on maximum diurnal organic loading, degree of treatment, and level of suspended solids 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 dissolved oxygen 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 of O_2/kg of peak BOD_5 applied to the aeration tanks with the exception of the extended aeration process, for which the value should be 1.5 to include respiration requirements.

Where nitrification is required or will occur, such as within the extended aeration process, the oxygen requirement for oxidizing ammonia must be added to the above requirement for carbonaceous BOD_5 removal and respiration requirements. The nitrogenous oxygen demand (NOD) should be taken as 4.6 times the diurnal peak hourly 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 the diurnal organic load variation while economizing on power input.

5.8.1.6.3.1. Aeration Equipment Selection

Evaluation of aeration equipment alternatives should include the following considerations:

1. Costs – capital, maintenance and operating;
2. Oxygen transfer efficiency;
3. Mixing capabilities;
4. Diffuser clogging problems;
5. Air pre-treatment requirements;
6. Total power requirements;
7. Aerator tip speed of mechanical aerators used with activated sludge systems;
8. Icing problems;
9. Misting problems; and
10. Cooling effects on aeration tank contents.

The size of the aeration tank for any particular adaptation of the process shall be determined by full-scale experience, pilot plant studies, or rational calculations based mainly on food to microorganism ratio and mixed liquor suspended solids levels. Other factors, such as size of treatment plant, diurnal load variations, and degree of treatment required, shall also be considered. In addition, temperature, pH, and reactor-dissolved oxygen shall be considered when designing for nitrification.

5.8.1.6.3.2. Diffused Air Systems

The design of the diffused air system to provide the oxygen requirements shall be done by either of the two methods described below in (1) and (2), augmented as required by consideration of items (3) through (8):

1. Having determined the oxygen requirements per Section 5.8.1.6.3, air requirements for a diffused air system shall be determined by use of any of the well known equations incorporating such factors as:
   a) Tank depth;
   b) Alpha factor of waste;
   c) Beta factor of waste;
   d) Certified aeration device transfer efficiency;
   e) Minimum aeration tank dissolved oxygen concentrations.
   f) Critical wastewater temperature; and
   g) Altitude of plant.

   In the absence of experimentally determined alpha and beta 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 percentage of industrial wastes should use a correspondingly lower percentage of clean water efficiency and shall have calculations submitted to justify such a percentage. The design transfer efficiency should be included in the specifications.

2. Normal 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 5.8.1.6.3), shall be considered to be 94 m^3 at standard conditions of pressure, temperature and humidity per kilogram of BOD_5 tank loading. For the extended aeration process the value shall be 125 m^3/kg of BOD_5.

3. To the air requirements calculated above shall be added air required for channels, pumps, aerobic digesters, filtrate and supernatant or other air-use demand.

4. The specified capacity of blowers or air compressors, particularly centrifugal blowers, should take into account that the air intake temperature may reach 40^{o}C or higher and the pressure may be less than normal. The specified capacity of the motor drive should also take into account that the intake air may be -30^{o}C or less and may require over-sizing of the motor or a means of reducing the rate of air delivery to prevent overheating or damage to the motor.

5. The blowers shall 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 shall also provide for varying the volume of air delivered in proportion to the load demand of the plant.

Aeration equipment shall be easily adjustable in increments and shall maintain solids suspension within these limits.

6. Diffuser systems shall be capable of providing for 200% of the design average oxygen demand. The air diffusion piping and diffuser system should be capable of delivering normal air requirements with minimal friction losses. Air piping systems should be designed such that total pressure loss from blower outlet (or silencer outlet where used) to the diffuser inlet does not exceed 3.4 kPa at average operating conditions. The spacing of diffusers shall be in accordance with the oxygen requirements through the length of the channel or tank, and shall be designed to facilitate adjustment of their spacing without major revision to air header piping. All plants employing less than four independent aeration tanks shall be designed to incorporate removable diffusers that can be serviced and/or replaced without dewatering the tank

7. Individual assembly units of diffusers should be equipped with control valves, preferably with indicator markings for throttling, or for complete shutoff. Diffusers in any single assembly shall have substantially uniform pressure loss.

8. Air filters shall be provided in numbers, arrangements, and capacities to furnish at all times an air supply sufficiently free from dust to prevent damage to blowers and clogging of the diffuser system used.

5.8.1.6.3.3. Mechanical Aeration Systems

5.8.1.6.3.3.1. Oxygen Transfer Performance

The mechanism and drive unit shall be designed for the expected conditions in the aeration tank in terms of the power performance. Certified testing shall verify mechanical aerator performance. Refer to applicable provisions of Section 5.8.1.6.3.2. In the absence of specific design information, the oxygen requirements shall be calculated using a transfer rate not to exceed 1.22 kg O_2/kW·hr in clean water under standard test conditions. Design transfer efficiencies shall be included in the specifications.

5.8.1.6.3.3.2. Design Requirements

The design requirements of a mechanical aeration system shall accomplish the following:

1. Maintain a minimum of 2.0 mg/L of dissolved oxygen in the mixed liquor at all times throughout the tank or basin;
2. Maintain all biological solids in suspension (for a horizontally mixed aeration tank system an average velocity of 0.3 m/s must be maintained;
3. Meet maximum oxygen demand and maintain process performance with the largest unit out of service;
4. Provide for varying the amount of oxygen transferred in proportion to the load demand on the plant; and
5. Provide that motors, gear housing, bearings, grease fittings, etc., be easily accessible and protected from inundation and spray as necessary for proper functioning of the unit.

5.8.1.6.3.3.3. Winter Protection
Where extended cold weather conditions occur, the aerator mechanism and associated structure shall be protected from freezing due to splashing. Due to high heat loss, the mechanism as well as subsequent treatment units shall be protected from freezing where extended cold weather conditions occur.

5.8.1.7. Process Definitions
The following are brief descriptions of a number of modifications of the activated sludge process.

5.8.1.7.1. Conventional Activated Sludge
The plug flow activated sludge process is a biological mechanism capable of removing 85 to 95% BOD 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 mixed liquor suspended solids (MLSS) in the initial part of the task. Plug flow configurations are often preferred when high effluent DO's are sought.

5.8.1.7.2. Complete Mix Activated Sludge
In a complete mix activated sludge 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. Because 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.

5.8.1.7.3. Step Aeration
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.

5.8.1.7.4. Contact 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 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% versus that required in an extended aeration system.

5.8.1.7.5. Extended Aeration
The extended aeration process used 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.

5.8.1.7.6. Oxidation Ditch
The oxidation ditch is a variation of the extended aeration process. The wastewater is pumped 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 0.2 to 0.37 m/s in the channel to prevent solids from settling.

Oxidation ditches use mechanical brush disk aerators, surface aerators, and jet aerator devices to aerate and pump the liquid flow.

5.8.1.7.7. 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 re-circulation rates (100 to 500%), and low hydraulic retention times. Depending on the excess sludge wasting procedure, 60 to 90% BOD removal is achieved for normal domestic wastes. This process is usually (but not necessarily) accomplished in "combined-tank" units.

5.8.1.7.8. 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.

5.8.1.8. Return Sludge Equipment
5.8.1.8.1. Return Sludge Rate
The minimum permissible return sludge rate of withdrawal from the final settling tank is a function of the concentration of suspended solids 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 tanks may be deleterious to both the aeration and sedimentation phases of the activated sludge process, the rate of sludge return expressed as a

percentage of the average design flow of wastewater should generally be variable between the limits set forth in Table 5.18.

The rate of sludge return shall be varied by means of variable speed motors, drives, or timers (small plants) to pump sludge at the above rates.

Table 5.18
Percentage of Average Design Flow

Type of Process	Minimum	Normal	Maximum
Plug Flow	25	30	100
Complete Mix	25	30	100
Carbonaceous Stage of Separate Stage	25		75
Stage Nitrification	25	50	75
Step Aeration	50	100	150
Contact Stabilization	50	100	150
Extended Aeration	50	100	150
Oxidation Ditch	50	50	200
High Rate Nitrification Stage of Separate Stage Nitrification	50		200

5.8.1.8.2. 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 pressure should be provided on pump suctions. Pumps should have at least 100 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 to facilitate their rapid and easy cleaning and provided other suitable standby measures are provided. Air lifts should be at least 100 mm in diameter.

5.8.1.8.3. Return Sludge Piping
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 when return sludge facilities are operating at normal return sludge rates. Suitable devices for observing, sampling, and controlling return activated sludge flow from each settling tank hopper shall be provided, as outlined in Section 5.6.1.3.2.5.

5.8.1.8.4. Waste Sludge Facilities
Waste sludge control facilities should have a maximum capacity of not less than 25% of the average rate of wastewater flow and function satisfactorily at rates of 0.5% of average wastewater flow or a minimum of 0.63 L/s, whichever is larger. Means for observing, measuring, sampling and controlling waste activated sludge flow shall be provided. Waste sludge may be discharged to the concentration or thickening tank, primary settling tank, sludge digestion tank, vacuum filters, or any practical combination of these units.

5.8.1.8.5. 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 tap water may be sprayed through these nozzles (either continuously or on a time clock on-off cycle) to physically break up the foam. Provision may be made to use antifoaming chemical agents into the inlet of the aeration tank or preferably into the spray water.

5.8.1.9. Measuring Devices
Devices should be installed in all plants for indicating flow rates of raw wastewater or primary effluent, return sludge, and air to each tank unit. For plants designed for wastewater flows of 450 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 flow rate to each aeration unit should be measured.

5.8.2. Rotating Biological Contactors

5.8.2.1. Applicability
The Rotating Biological Contactor (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 rotating biological contactor (RBC) process should include:
1. Raw sewage amenability to biological treatment;
2. Pretreatment effectiveness including scum and grease removal;
3. Expected organic loadings, including variations;
4. Expected hydraulic loadings, including variations;
5. Treatment requirements, including necessary reduction of carbonaceous and/or nitrogenous oxygen demand;
6. Sewage characteristics, including pH, temperature, toxicity, nutrients;
7. Maximum organic loading rate of active disc surface area; and
8. Minimum detention time at maximum design flow.

5.8.2.2. Winter Protection
Wastewater temperature affects rotating contactor performance. Year-round operation in colder climates 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.

Enclosures shall be constructed of a suitable corrosion resistant material. Windows or simple louvred mechanisms, which can be opened in the summer and closed in the winter, shall 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 RBC's are contained within a building provided with interior access for personnel.

5.8.2.3. Flow Equalization

For economy of scale, the peaking factor of maximum flow to average daily flow should not exceed 3. Flow equalization should be considered in any instance where the peaking factor exceeds 2.5.

5.8.2.4. Operating Temperature

The temperature of wastewater entering any RBC should not drop below 13°C unless there is sufficient flexibility to decrease the hydraulic loading rate or the units have been increased in size to accommodate the lower temperature. Otherwise, insulation or additional heating must be provided to the plant.

5.8.2.5. Design Flexibility

Adequate flexibility in process operation should be provided by considering one or more of the following:

1. Variable rotational speeds in first and second stages;
2. Multiple treatment trains;
3. Removable baffles between all stages;
4. Positive influent flow control to each unit or flow train;
5. Positively controlled alternate flow distribution systems;
6. Positive airflow metering and control to each shaft when supplemental operation or air drive units are used; and
7. Recirculation of secondary clarifier effluent.

5.8.2.6. Hydrogen Sulphide

When higher than normal influent or sidestream hydrogen sulphide concentrations are anticipated, appropriate modifications in the design should be made.

5.8.2.7. Pretreatment

RBC's must be preceded by effective settling tanks equipped with scum and grease collecting devices unless substantial justification is submitted for other pretreatment devices, which provide for effective removal of grit, debris and excessive oil or grease prior to the RBC units. Bar screening or comminution are not suitable as the sole means of pretreatment.

5.8.2.8. Unit Sizing

The Designer of an RBC system shall 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.

Unit sizing shall 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:

1. Design flow rate and influent waste strength;
2. Percentage of BOD to be removed;
3. Media arrangement, including number of stages and unit areas in each stage;
4. Rotational velocity of the media;
5. Retention time within the tank containing the media;
6. Wastewater temperature; and
7. Percentage of influent BOD, which is soluble.

In addition to the above parameters, loading rates for nitrification will depend upon influent total kjeldahl nitrogen (TKN), pH, and the allowable effluent ammonia nitrogen concentration.

5.8.2.9. Hydraulic Loading

Hydraulic loading to the RBC's should range from 75 to 155 L/m^{2}d of media surface area without nitrification, and 30 to 80 L/s with nitrification.

5.8.2.10. Organic Loading

The RBC process is approximately first order with respect to BOD removal; i.e. for a given hydraulic loading (or retention time) a specific percent BOD reduction will occur, regardless of the influent BOD concentration. However, BOD 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^{2}d or 0.012 to 0.02 kg BOD soluble/m^{2}d.

Loadings in the higher end of these ranges will increase the likelihood of developing problems such as heavier than normal biofilm thickness, depletion of dissolved oxygen, 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.

5.8.2.11. 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 is 0.042 L/m^2, which takes into account wastewater, displaced by the media and attached biomass. The use of tank volumes in excess of 0.042 L/m^2 does not yield corresponding increases in treatment capacity when treating wastewater in this concentration range.

5.8.2.12. Detention Time

Based on a tank volume of 0.042 L/m^2, the detention time in each RBC stage should range between 40 to 120 minutes without nitrification, and 90 to 250 minutes with nitrification.

5.8.2.13. Media Submergence and Clearance

RBC's 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 so as to maintain sufficient bottom velocities to prevent solids deposition in the tank.

5.8.2.14. Design Considerations

5.8.2.14.1. 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.

5.8.2.14.2. Tankage

RBC units may be placed in either steel or concrete tankage with baffles, when required, and constructed of a variety of materials. The design of the tankage must include:

1. Adequate structural support for the RBC and drive unit;
2. Elimination of the "dead" areas;
3. Satisfactory hydraulic transfer capacity between stages of units; and
4. 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 one foot 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.

5.8.2.14.3. 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.

5.8.2.14.4. Shaft Rotational Velocity

The peripheral velocity of a rotating shaft should be approximately 18 m/min for mechanically driven shaft, and between 9 and 18 m/min for an air driven shaft. Provision should also be made for rotational speed control and reversal.

5.8.2.14.5. Biomass Removal

A means for removing excess biofilm growth should be provided, such as air or water stripping, chemical additives, rotational speed control/reversal, etc.

5.8.2.14.6. Dissolved Oxygen Monitoring

First-stage dissolved oxygen (DO) monitoring should be provided. The RBC should be able to maintain a positive DO level in all stages.

5.8.2.14.7. Supplemental Air

Periodic high organic loadings may require supplemental aeration in the first stage to promote sloughing of biomass.

5.8.2.14.8. 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 facility 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.

5.8.2.14.9. Re-circulation

Consideration should be given to providing re-circulation of RBC effluent flow. This may be necessary during initial start-up and when the inflow rate is reduced to extremes.

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 biogrowth to enter the endogenous respiration phase where portions of the biogrowth become the food source or substrate for other portions of the biogrowth. If this condition lasts long enough, all of the biogrowth will eventually be destroyed. When this condition is allowed to exist, the RBC process does not have adequate biogrowth 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 biogrowth and, as a secondary benefit, help stabilize and reduce the sludge.

When any new facility is first started, the biogrowth is slow to establish. If it is desired to build up the biogrowth 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 biogrowth will develop with only minimal removal of the organic load. By recycling, the unused organic load again becomes available to the biogrowth. As the biogrowth develops, the recycle rate should be reduced, with new inflow added to increase the organic load. As the biogrowth develops further, the recycle is eventually reduced to zero with all of the inflow being the normal RBC influent.

5.8.2.14.10. 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. Stop motion detectors, rpm indicators and clamp-on ammeters are also potentially useful monitoring instruments.

Therefore, load cells shall be provided for all first and second stage shafts. Load cells for all other shafts in an installation are desirable.

5.8.2.14.11. 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.

5.8.2.14.12. 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 facility. 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.

5.8.2.14.13. 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.

5.8.2.14.14. 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.

5.8.3. Sequencing Batch Reactor (SBR)

The Sequencing Batch Reactor (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; and
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:

1. Fill Method;
2. Hydraulic Control Systems;
3. Aeration Control Systems;
4. Method of Decant;
5. Sizing of Disinfection Equipment for Decant Flows; and
6. 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 the DOEC.

5.8.3.1. 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).

5.8.3.2. Continuous Influent Systems

Continuous feed-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 and also spreads 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. Because CFID systems generally operate on the basis of preset time cycles, TWL varies for each cycle as a function of the influent flow for that particular cycle. The actual effluent flow rate 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 2:1 to 4:1 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.

5.8.3.3. Intermittent Influent Systems

IFID 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. Systems should 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 preset length of time) or by volume (that is, fill until the water level rises a fixed amount). 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 period (typically 1 to 3 hours). Again, the time devoted to reaction in any given cycle may automatically be changed as a function of influent flow rate. At the completion of the reaction phase, aeration and any supplemental mixing are 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 the time period 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 flow rate during discharge has the potential to be several times higher than the influent flow rate. Discharge flow rates 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.

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; and
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.

5.8.3.4. Sequencing Batch Reactor Equipment

5.8.3.4.1 Process Control

The programmable logic controller (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 desktop 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, and idle times. In addition, the design engineer may develop additional software to interface to PLC to a desktop computer for graphic presentation of process operation to the operator and generation of archive date and compliance reports.

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 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 owner is limited to maintenance and repair functions that are well within the capability of a competent electrician.

5.8.3.4.2. 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.

5.8.3.4.3. 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 tear, 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.

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.

5.8.3.4.4. Solids Wasting

The wasting of both aerated mixed liquor suspended solids (MLSS) 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.

5.8.3.4.5. Aeration/Mixing Systems

A variety of aeration and mixing systems are in use with SBRs. These include jet aeration, fine- and course-bubble aeration, 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.

5.9. Wastewater Treatment Ponds (Lagoons)

This section deals with generally used variations of treatment lagoons to achieve secondary treatment including controlled-discharge lagoon systems, flow-through lagoon systems and aerated lagoon systems. Lagoons utilized for equalization, percolation, evaporation and sludge storage will not be discussed in this section.

5.9.1. Supplement to the Pre-design Report

The Pre-design report shall 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 sub-sections detail the information that must be submitted in addition to that required in Section 2.

5.9.1.1. Supplementary Field Survey Data

5.9.1.1.1. Location of Nearby Facilities

The location and direction of all residences, commercial developments, parks, recreational areas, and water supplies within l.6 km of the proposed lagoon shall be included in the Pre-design report.

5.9.1.1.2. Land Use Zoning

Land use zoning adjacent to the proposed lagoon site shall be included.

5.9.1.1.3. Site Description

A description, including maps showing elevations and contours of the site and adjacent area shall be provided. Due consideration shall be given to additional treatment units and/or increased waste loadings in determining land requirements.

5.9.1.1.4. Location of Field Tile

The location, depth, and discharge point of any field tile in the immediate area of the proposed site shall be identified.

5.9.1.1.5. 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 lagoon shall also be provided. At least one boring shall be a minimum of 7.6 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 shall be filled and sealed. The permeability characteristics of the lagoon bottom and lagoon seal materials shall also be studied (see Section 5.9.6.2).

5.9.1.1.6. Sulphate Content of Water Supply

Sulphate content of the basic water supply shall be determined.

5.9.1.1.7. Percolation Rates
Data demonstrating anticipated percolation rates at the elevation of the proposed lagoon bottom shall be included.

5.9.1.1.8. Well Survey
A pre-construction survey of all nearby wells (water level and water quality) is mandatory.

5.9.2. Location

5.9.2.1. Distance From Habitation
A lagoon site should be located as far as practicable, with a minimum of 150 m from isolated habitation and 300 m from built up areas or areas, which may be built up within a reasonable future period. Consideration should be given to site specifics such as topography, prevailing winds, forests, etc.

A minimum distance of 100 m from public roads and highways is also recommended. Aerated stabilization basins separation distances shall be considered the same as mechanical plants.

5.9.2.2. Prevailing Winds
If practicable, lagoons should be located so that local prevailing winds will be in the direction of uninhabited areas.

5.9.2.3. Surface Runoff
Location of lagoons in watersheds receiving significant amounts of stormwater runoff is discouraged. Adequate provision must be made to divert stormwater runoff around the lagoons and protect pond embankments from erosion.

5.9.2.4. Groundwater Pollution
Existing wells, which serve as drinking water sources should be protected from health hazards. Possible travel of pollutants through porous soils and fissured rocks should be objectively evaluated to safeguard the wells. A lagoon shall be located as far as practicable, 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 lagoon 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 DOEC.

A minimum of 1.5 m between the bottom of the lagoon and bedrock is recommended, however, less separation may be acceptable when supported by appropriate hydrogeological and engineering designs/investigations upon acceptance of the DOEC.

5.9.2.5. Protection of Surface Water Supplies

Lagoons shall be located downhill, downstream and remote from all sources of surface water supplies (lakes and rivers). The minimum distances outlined in Table 5.19 shall be employed as the criteria:

Table 5.19
Minimum Distance Criteria

Minimum Distance from a Lake or River to the Centre of a Dyke of a Proposed Lagoon	Remarks
120 m	Lined lagoon, pervious soil
75 m	Lined lagoon, impervious soil

5.9.2.6. Geology

Lagoons shall 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).

A minimum separation of 3.0 m between the lagoon bottom and any bedrock formation is recommended.

5.9.2.7. Floodplains

A lagoon shall not be located within the 100-year floodplain.

5.9.3. Definitions

Aerobic Lagoon - Aerobic lagoons are shallow basins used for wastewater treatment. The organic contaminants in the wastewater are degraded by aerobic and facultative bacteria. The lagoons characteristically receive a light organic loading. They are used primarily to achieve additional organic removal following conventional wastewater treatment. Dissolved oxygen is furnished by oxygen transfer between the air and water surface, and by photosynthetic algae. The amount of oxygen supplied by natural surface re-aeration depends largely on wind-induced turbulence.

Facultative Lagoons - The facultative lagoon is divided into an aerobic layer at the top and an anaerobic layer on the bottom. The aerobic layer is generated by algae, which produce oxygen by photosynthesis. Settleable solids are permitted to accumulate on the pond bottom, and are broken down anaerobically. Waste stabilization is accomplished by a combination of anaerobic, aerobic, and a preponderance of facultative organics interacting with the wastewater.

Aerated Lagoons - An aerated lagoon may be aerated aerobic (completely mixed) or aerated facultative. It does not depend on algae and sunlight to furnish DO or bacterial respiration, but instead uses diffusers or other mechanical aeration devices to transfer the major portion of oxygen and to create some degree of mixing. Because of the mixing, removal of suspended solids in the lagoon effluent is an important consideration. Aerated lagoons can be described as heavily loaded oxidation basins, or very lightly loaded activated sludge systems. The microorganisms responsible for the organic breakdown tend to be similar to those found in activated sludge systems.

Aerated lagoons may be used in series with aerobic lagoons. In such cases, the primary purpose of the lagoon without aeration is for solids removal.

5.9.4. Application, Advantages and Disadvantages of Different Lagoon Types

Table 5.20 details the appropriate application of various lagoon types, as well as summarizes the advantages and disadvantages related to each type.

Table 5.20
Application, Advantages and Disadvantages of the Different Lagoon Types

Parameter	Unaerated Aerobic	Facultative	Aerated	
			Aerobic	Facultative
Application	Nutrient Removal; Treatment of Soluble Organic Wastes; Secondary Effluents	Treatment of raw domestic and industrial wastes	Treatment of raw domestic and industrial wastes	Treatment of raw domestic and industrial wastes
Advantages	Low operating and maintenance costs	Low operating and maintenance costs	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 and operating costs; high solids in effluent; foaming	Maintenance and operation costs; foaming

5.9.5. Basis of Design

5.9.5.1. Lagoons

5.9.5.1.1. Holding Capacity Requirements

Before the design of a lagoon system can be initiated, the designer shall determine the following:

1. Whether the lagoon can be continuously discharged or must operate on a fill-and-draw basis;

2. The period of the year if any, when discharge will not be permitted;

3. What discharge rates will be permitted with fill-and-draw lagoons and what, if any, provision must be made for controlling effluent discharge rates in proportion to receiving stream flow rates; and

4. What the minimum time for discharge of lagoon cell contents should be for fill-and-draw systems.

The holding capacity of lagoons shall be based upon average daily sewage flow rates, making a special allowance for net precipitation entering the cells.

5.9.5.1.2. Area and Loadings

One hectare of water surface should be provided for each 250-design population or population equivalent. In terms of BOD, 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 DOEC. Due consideration shall be given to possible future municipal expansion 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.

Where substantial ice cover may be expected for an extended period, it may be desirable to operate the facility to completely retain wintertime flows. Design variables such as lagoon depth, multiple units, detention time and additional treatment units must be considered with respect to applicable standards for BOD_5, total suspended solids (TSS), fecal coliforms, dissolved oxygen (DO) and pH.

5.9.5.1.3. Flow Distribution

The main inlet sewer or forcemain should terminate at a chamber, which permits hydraulic and organic load splitting between the lagoon cells. The ability to introduce raw sewage to all cells is desirable, but as a minimum, there must be a capability to divide raw sewage flows between enough cells to reduce the BOD_5 loading to 22 kg BOD_5/ha·day, or less. The inlet chamber should be provided with a lockable aluminum cover plate or grating, divided into small enough sections to permit easy handling.

5.9.5.1.4. Typical Performance Potentials

Atlantic region environmental conditions are expected to facilitate the following performance of lagoons treating typical domestic wastes:

Winter efficiency = 70% BOD removal
Summer efficiency = 80% BOD removal
Organic Load = 22 kg BOD_5/ha·day
Liquid depth = 1.5 to 1.8 m
Suspended Solids removal = 80% but may decrease with increasing algal concentrations

5.9.5.1.5. Controlled - Discharge Lagoons

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.

5.9.5.1.6. Flow-Through Lagoons
At a minimum, primary cells shall provide adequate detention time to maximize BOD removal. Secondary cells should then be provided for additional detention time with depths to 2.0 m to facilitate both solids and coliform reduction.

5.9.5.1.7. Tertiary Lagoon
When lagoons are used to provide additional treatment for effluents from existing or new secondary sewage treatment works, the DOEC will, upon request, establish BOD loadings for the lagoon after due consideration of the efficiencies of the preceding treatment units.

5.9.5.2. Aerated Lagoons
Aerated lagoons can be either aerobic or facultative. An aerated aerobic lagoon contains dissolved oxygen through the whole system with no anaerobic zones. The lagoon shape and the aerating power provide complete mixing. The aerated facultative lagoon provides a partially mixed condition, which will cause an anaerobic zone to develop at the bottom as suspended solids settle due to low velocity in the system.

5.9.5.2.1. Aerated Aerobic Lagoons
In general, an aerated lagoon can be classified as an aerobic lagoon (complete mixed) if the mechanical aeration power level is above six watts per cubic meters of maximum storage. Aerated aerobic lagoons 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 suspended solids removal processes such as a clarifier must follow aerated aerobic treatment, to insure compliance with suspended solids discharge requirements. In most cases, a minimum detention time of one day is required to achieve solids separation. Algae growth should be limited by controlling the hydraulic detention time to two days or less. Water depth of not less than 1.0 m shall 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.

5.9.5.2.2. Aerated Facultative Lagoons
Aerated facultative lagoons should be designed to maintain a minimum of 2.0 mg/L of dissolved oxygen (DO) in the upper zone of the liquid. The aeration system must be able to transfer up to 1.0 kg of oxygen per kg of 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^3day.

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^2d. If multiple aerated facultative cells are used, all cells following the first one shall have diminished aeration capacity to permit additional settling. Whenever possible, provisions should be provided for re-circulating part (5-10%) of the final aeration cell effluent back into the influent in order to maintain a satisfactory mix of active microorganisms.

5.9.5.2.3. Design Parameters

5.9.5.2.3.1. Detention Time

The mean cell residence time of an aerated lagoon should ensure that the suspended micro-organisms have adequate detention time to transform non-settling and dissolved solids into settleable solids, an adequate factor of safety is provided for periods of high hydraulic loading and that the detention time in the aerated lagoon is controlled by the rate of metabolism during the coldest period of the year.

As a minimum, the detention time should reflect 85% BOD_5 removal from November to April, being based on good and efficient operation of the aeration equipment. For the development of final design parameters, it is recommended that actual experimental data be developed; however, the aerated lagoon system design for minimum detention time may be estimated using the following formula:

$$t = \frac{E}{2.3K1 \times (100 - E)}$$

where: t = detention time (days);

E = percent of BOD5 to be removed in an aerated lagoon; and

K1 = reaction coefficient, aerated lagoon, base 10. For normal domestic sewage, the K1 value may be assumed to be 0.12/d at 20°C and 0.06/d at 1°C.

The reaction rate coefficient for domestic sewage which includes some industrial wastes, other wastes and partially treated sewage must be determined experimentally for various conditions which might be encountered in the aerated lagoons. Conversion of the reaction rate coefficient at other temperatures shall be made based on experimental data. Additional storage volume should be considered for sludge and ice cover.

5.9.5.2.3.2. Oxygen Requirement

Oxygen requirements generally will depend on the BOD loading, the degree of treatment and the concentration of suspended solids to be maintained. Aeration equipment shall be capable of maintaining a minimum dissolved oxygen level of 2.0 mg/L in the lagoons at all times. The oxygen requirements should meet or exceed the peak 24-hour summer loadings. A safety factor of up to 2 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. Suitable protection from weather shall be provided for electrical control.

5.9.5.3. Industrial Wastes

Consideration shall be given to the type and effects of industrial wastes on the treatment process. In some cases it may be necessary to pretreat industrial or other discharges.

Industrial wastes shall not be discharged to lagoons without assessment of the effects such substances may have upon the treatment process or the discharge requirements of the DOEC.

5.9.5.4. Multiple Units

At a minimum, a lagoon system should consist of 2 cells designed to facilitate both series and parallel operations. The maximum size of a lagoon cell should be 5 ha. A one-cell system may be utilized in very 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. In addition, the ability to discharge the influent waste load to a minimum of 2 cells and/or all primary cells in the system should be provided.

Requirements for multiple units in an aerated lagoon system shall be similar to those in an activated sludge system, including requirements for back-up aeration equipment.

5.9.5.5. 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. In no case should lagoon depths be less than 0.6 m.

5.9.5.5.1. Controlled-Discharge Lagoons

The maximum water depth shall be 1.8 m in primary cells. Greater depths in subsequent cells are permissible although supplemental aeration or mixing may be necessary.

5.9.5.5.2. Flow-Through Lagoons

Maximum normal liquid depth should be 1.5 m.

5.9.5.5.3. Aerated Lagoon Systems

In general, normal water depths vary from 1.2 to 3.6 m when using surface aerators, however, consideration should be given to depths of up to 5.0 m to minimize surface heat losses.

5.9.5.6. Lagoon Shape

Square cells are preferred to long narrow rectangular cells. Round, square or rectangular lagoons with a length not exceeding three times the width are considered most desirable. The long dimension of any pond should not align with the prevailing wind direction. No islands, peninsulas or coves shall be permitted. Dykes should be rounded at corners to minimize accumulations of floating materials. Common-wall dyke construction, wherever possible, is strongly encouraged.

5.9.5.7. Additional Treatment

Consideration should be given in the design stage to the utilization of additional treatment units as may be necessary to meet applicable discharge standards.

5.9.6. Lagoon Construction Details

5.9.6.1. Embankments and Dykes

5.9.6.1.1. Materials

Embankments and dykes shall be constructed of relatively impervious material and compacted to at least 90% Standard Proctor Density to form a stable structure. Vegetation and other unsuitable materials shall be removed from the areas where the embankment is to be placed.

5.9.6.1.2. Top Width

The minimum dyke width shall be 3.0 m to permit access of maintenance vehicles.

5.9.6.1.3. Maximum Slopes

Inner and outer dyke slopes shall not be steeper than 1 vertical to 3 horizontal (1:3).

5.9.6.1.4. Minimum Slopes

Inner slopes should not be flatter than 1 vertical to 4 horizontal (1:4). Flatter slopes can be specified for larger installations because of wave action but have the disadvantage of added shallow areas being conducive to emergent vegetation. Outer slopes shall be sufficient to prevent surface runoff from entering the ponds.

5.9.6.1.5. Freeboard

Minimum freeboard shall be 1.0 m. For very small systems, 0.6 m may be acceptable.

5.9.6.1.6. Erosion Control

1. **Outer Dykes** - The outer dykes shall have a cover layer of at least 100 mm of fertile topsoil to promote establishment of an adequate vegetative cover wherever riprap is not utilized. Adequate vegetation shall 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 to severe flooding of a watercourse.

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 plant owner. The inner dykes shall 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 riprap or an acceptable equal shall 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.

3. **Top of Embankment** - The top of the embankment used for access around the perimeter of the dykes shall have a cover layer of at least 300 mm of cover material similar to the one described in Section 5.9.6.1.6 (2).

4. **Additional Erosion Protection** - Riprap 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.

5. **Erosion Control During Construction** - Effective site erosion control shall be provided during construction as required by this DOEC.

6. **Seeding** - The dykes shall have a cover layer of at least 100 mm of fertile topsoil to promote establishment of an adequate vegetative cover wherever riprap is not utilized. Prior to pre-filling (in accordance with Section 5.9.6.4), adequate vegetation shall be established on dykes from the outside toe to 0.6 m above the pond bottom on the interior as measured on the slope. Perennial-type, low-growing, spreading grasses that minimize erosion and can be mowed are most satisfactory for seeding on dikes. In general, alfalfa and other long-rooted crops should not be used for seeding since the roots of this type are apt to impair the water holding efficiency of the dikes.

5.9.6.2. Lagoon Bottom and Liners

5.9.6.2.1. Location

A minimum separation of 3.0 m between the cell bottom and bedrock is recommended. Cell bottoms should be located at least 1.2 m above the high groundwater level, in order to prevent inflow and/or liner damage.

5.9.6.2.2. 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.

5.9.6.2.3. Vegetation

The bottom shall be cleared of vegetation and debris. Organic material thus removed shall 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 5.9.6.1.6. (2).

5.9.6.2.4. Soil

Soil used in constructing the lagoon bottom (not including liner) and dyke cores shall 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.

5.9.6.2.5. Liner
Lagoons shall 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 shall 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.

5.9.6.2.6. Seepage Control Criterion for Clay Liners
The seepage control criterion for municipal wastewater lagoons and aerated lagoons utilizing clay liners specifies a maximum hydraulic conductivity, K, for the lagoon liner as a function of the liner thickness, L, and water depth, D, by the equation:

$$Maximum\,K\left(\tfrac{m}{s}\right)=\frac{4.6\times10^{-8}\left(\tfrac{m}{s}\right)\times L(m)}{D(m)+L(m)}$$

For example, a compacted clay liner that is 0.5 m thick must have a hydraulic conductivity of about 1.3 x 10^{-8} m/s (1.3 x 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 cubic meters per day per hectare at a water depth of 1.2 metres.

5.9.6.2.7. Seepage Control Criterion for Synthetic Liners
For synthetic liners, seepage loss through the liner shall not exceed the quantity equivalent through an adequate soil liner. For liner durability the minimum liner thickness for a HDPE liner shall be 1.5 mm (60 mil). The liner shall be underlain by a sand layer with a minimum thickness of 150 mm. Special consideration should be given to problems associated with bedding movement on berm slopes. Provision must be made to check integrity or possible leakage of liner by means of lysimeters, sampling stations or access to sub-drainage collection system.

5.9.6.2.8. 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.

5.9.6.3. Design and Construction Procedures for Clay Liners

5.9.6.3.1. 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.

5.9.6.3.2. Liner Thickness

Recommended minimum compacted clay liner thickness 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 m is suggested to allow economic and proper placement and compaction of the clay using large earth-moving equipment.

5.9.6.3.3. 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), i.e.: K (design) = K (in situ) = 10 x 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 5.9.6.2.6. 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.

Permeability tests shall 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:

1. Laboratory tests on samples from below the proposed bottom of the stabilization basin and from the material to be used in the dykes; and

2. Field seepage tests. These may be conducted in the following way. A pit shall be dug to the level of the proposed stabilization basin bottom and the bottom of the dug hole carefully cleaned. At least one test shall be conducted for every two hectares of stabilization basin area. A pipe with an internal diameter of at least 0.2 m and length of at least 1.2 m shall be carefully placed in a vertical position resting on the bottom of the hole. The hole shall be backfilled around the outside of the pipe to a height of 1.o m with carefully tamped soil. Particular care should be given to tamping of soil near the bottom.

The pipe shall be filled with water to a depth of 1.2 m. The water must be placed in the pipe gently so as not to disturb the soil at the bottom.

The drop in water level from a head of 1.2 m shall be recorded for each of at least three 24-hour periods, or until the readings become consistent. (Level shall be re-adjusted to 1.2 m at the beginning of each 24-hout period).

5.9.6.3.4. 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.

5.9.6.3.5. Liner Material Placement and 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 etc., should either be 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.

5.9.6.3.6. 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 5.9.6.3.3). 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 (say 30 x 30 m to 60 x 60 m grids for large ponds and at closer spacing for small lagoons) 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 5.9.6.3.3). 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.

5.9.6.3.7. 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, for stockpiling, removal of undesirable materials and possible additional borrowing outside of the cut area.

5.9.6.4. Prefilling

Prefilling the pond should be considered in order to protect the liner, to prevent weed growth, to reduce odour and to maintain moisture content of the seal. However, the dykes must be completely prepared as described in Section 5.9.6.1.6 before the introduction of water.

5.9.6.5. Influent Lines

5.9.6.5.1. Material

Generally accepted material for underground sewer construction will be given consideration for the influent line to the lagoon. 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.

5.9.6.5.2. Manhole

A manhole or vented cleanout wye shall be installed prior to entrance of the influent line into the primary cell and shall be located as close to the dike as topography permits. Its invert shall be at least 150 mm above the maximum operating level of the lagoon and provide sufficient hydraulic pressure without surcharging the manhole.

5.9.6.5.3. Surcharging

The design and construction of influent piping shall insure that where surcharging exists, due to the head of the lagoon, no adverse effects will result. These effects shall include basement flooding and overtopping of manholes.

5.9.6.5.4. Forcemains

Forcemains terminating in a sewage lagoon should be fitted with a valve immediately upstream of the lagoon.

5.9.6.5.5. Flow Distribution

Flow distribution structures shall be designed to effectively split hydraulic and organic loads equally to primary cells.

5.9.6.5.6. Location

Influent lines shall be located along the bottom of the lagoon so that the top of the pipe is just below the average elevation of the lagoon seal, however, the pipe shall have adequate seal below it. The use of an exposed dyke to carry the influent line to the discharge points is prohibited.

5.9.6.5.7. Point of Discharge

The influent line to a square single celled lagoon should be essentially centre discharging. Each square cell of a multiple celled lagoon operated in parallel shall 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 lagoons 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 lagoon. Consideration should be given to multi-influent discharge points for primary cells of 5 ha or larger.

All aerated cells shall 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 lagoons the inlet pipe may go directly through the dyke and end at the toe of the inner slope.

5.9.6.5.8. Influent Discharge Apron

Inlet pipes should terminate with an upturned elbow, with the pipe extending 450 mm above the cell bottom. The end of the discharge line shall 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 1.0 m^2 shall be provided.

5.9.6.5.9. Pipe Size

The influent system shall be sized to permit peak raw sewage flow to be directed to any one of the primary cells. Influent piping should provide a minimum scouring velocity of 0.6 m/s.

5.9.6.6. Control Structures and Interconnecting Piping

5.9.6.6.1. Structure

Where possible, facilities design shall 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.

As a minimum, control structures shall be:

1. Accessible for maintenance and adjustment of controls;
2. Adequately ventilated for safety and to minimize corrosion;
3. Locked to discourage vandalism;
4. Contain controls to permit water level and flow rate control, complete shutoff, and complete draining;
5. Constructed of non-corrodible materials (metal-on-metal contact in controls should be of similar alloys to discourage electrochemical reactions); and
6. 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 preset to stop flows at any pond elevation.

5.9.6.6.2. Piping

All piping shall be of ductile iron or other acceptable material. The piping shall not be located within or below the liner. Pipes should be anchored with adequate erosion control.

1. Drawdown Structure Piping:
    a) **Submerged Takeoffs** - For lagoons designed for shallow or variable depth operations, submerged take offs are recommended. Intakes shall be located a minimum of 3.0 m from the toe of the dyke and 0.6 m from the top of the seal, and shall employ vertical withdrawal;

    b) **Multi-Level Takeoffs** - For lagoons that are designed deep enough to permit stratification of lagoon content, multiple takeoffs are recommended. There shall be a minimum of 3 withdrawal pipes at different elevations. The bottom pipe shall conform to a submerged takeoff. The others should utilize horizontal entrance. Adequate structural support shall be provided.

c) **Surface Takeoffs** - For use under constant discharge conditions and/or relatively shallow lagoons 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.

d) **Maintenance Drawdown** - All lagoons shall have a lagoon drain to allow complete emptying, either by gravity or pumping, for maintenance. These should be incorporated into the above-described structures. In aerated lagoons where a diffused air aeration system and submerged air headers are used, provision should be made to drain each lagoon (independently of others) below the level of the air header.

e) **Emergency Overflow** - All cells shall be provided with an emergency overflow system, which overflows when the liquid reaches within 0.6 m of the top of the berms.

5.9.6.6.3. Hydraulic Capacity
The hydraulic capacity for continuous discharge structures and piping shall allow for at least the expected future peak sewage flows.

The hydraulic capacity for controlled-discharge systems shall permit transfer of water at a minimum rate of 150 mm of lagoon water depth per day at the available head.

5.9.6.6.4. 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 lagoon bottom to minimize need for erosion control measures and should be located as near the dividing dyke as construction permits. Interconnection piping shall enable parallel or series flow patterns between cells.

5.9.6.6.5. Location
The outlet structure and the inter-connecting pipes should be located away from the corners where floating solids accumulate, and on the windward side to prevent short-circuiting.

5.9.7. Miscellaneous

5.9.7.1. Fencing
The lagoon area shall be enclosed with an adequate fence to prevent entering of livestock and discourage trespassing. Fencing should not obstruct vehicle traffic on top of the dyke. A vehicle access gate of sufficient width to accommodate mowing equipment shall be provided. All access gates shall be provided with locks.

5.9.7.2. Access

An all-weather access road shall be provided to the lagoon site to allow year-round maintenance of the facility.

5.9.7.3. Warning Signs

Appropriate permanent signs shall be provided along the fence around the lagoon to designate the nature of the facility and advise against trespassing. At least one sign shall be provided on each side of the site and one for every 150 m of its perimeter.

5.9.7.4. Flow Measurement

Provisions for flow measurement shall be provided on the outlet. Safe access to the device should be made to permit safe measurement.

5.9.7.5. Groundwater Monitoring

An approved system of wells or lysimeters may be required around the perimeter of the lagoon site to facilitate groundwater monitoring. The need for such monitoring will be determined on a case-by-case basis.

5.9.7.6. Pond Level Gauges

Pond level gauges shall be provided.

5.9.7.7. Service Building

A service building for laboratory and maintenance equipment shall be provided, if required.

5.9.7.8. Liquid Depth Operation

Optimum liquid depth is influenced to some extent by lagoon 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.

5.9.7.9. Pretreatment and Post-Treatment

The wastewater shall be treated by bar screens and grit removal before entering the lagoon. The treated effluent shall be disinfected, as per regulatory requirements, prior to discharging into the receiving water.

5.10. Kikuth Bioreactor (Phytoklare)

The Kikuth Bioreactor or Phytoklare wastewater treatment system (Root Zone or Reed Bed) is a process that can be used in areas where a conventional septic tank system cannot be used and in situations where a mechanical wastewater treatment system may not be practical or feasible.

The system is a proprietary one whose basic makeup consists of primary treatment (septic tank, settling tank), influent and effluent piping and a subsurface wetland treatment system consisting of one or more cells with an impermeable liner, specialized soil matrix, specific plants and a disinfection system if required.

The system is a passive system with no moving parts. Because the up front treatment consists of primary treatment only, then consideration has to be given to the impact that extraneous flows (e.g. wet weather) may have on the wetland component as excessive solids carry over could lead to blockage problems of the subsurface system. Also, this system may not be appropriate for areas where extreme cold temperatures are encountered.

5.11. BMS “Blivet”

The BMS “Blivet” is a patented package wastewater treatment system that incorporates a combination of technologies similar to the rotating biological contactor and conventional activated sludge and sedimentation processes. The treatment sections are composed of primary sedimentation utilizing lamella plates, aeration sections with rotating drums with a very large surface area inside (patented design). The effluent is drawn in via holes in its periphery. Once inside it passes through the maze of surfaces. The combined effect of being actively mixed with air and passing over the bacterial surfaces provides effective aerobic treatment. Then the effluent passes through a secondary settlement area using lamella plates and final discharge. Normal treatment standards are 20mg/L BOD and 30 mg/L SS. The manufacturer suggests that the process is particularly suitable for population equivalents of between 10 to 3000 residents. They are used in areas not connected to mains sewers such as hotels, golf clubs, country clubs, holiday resorts, housing developments, condominiums, small townships and military and civil institutions.

5.12. Biogreen

The Biogreen system is designed primarily to provide wastewater treatment for individual homes or small communities and is comprised of seven chambers. The first two chambers are settling tanks, which remove sediment, debris and floating material from the influent. The first settling tank is larger than conventional systems and acts as a buffer for controlling the organic load through the unit. This is often a serious problem in conventional systems but is easily dealt with in a Biogreen system. The water passes from the settling tanks to the fermentation tank, which is filled with filters, and most of the anaerobic digestion of the sewage occurs here.

The next stage of the treatment process is aeration. The flow from the fermentation tank is controlled by a control valve to the first of two aeration tanks. This minimizes the shock loading which may occur in the aeration chamber. A draft tube is located in the middle of each aeration chamber, into which air is injected. This creates ideal circulation and maintains an adequate DO concentration. A DO gradient from zero to saturation is created, which allows for a longer food chain for nutrient digestion. This reduces sludge production as compared to existing methods. The final step is the effluent holding tank, which removes suspended solids before discharge.

5.13. 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 5.4.3.2. A number of 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 this manual the design data presented is fairly general in nature. A description of these systems mainly describing their application and typical loading rates is provided here. New treatment schemes may be added to the main section of this chapter when sufficient and adequate design data becomes available. These additions will be noted in the revision record.

5.13.1. 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 suspended solids 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. BAFs are designed either as co-current backwash or counter current 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 counter current 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 counter current to the wash water flow.

5.13.1.1. Design Features

The granular media bed for both designs typically is 3 to 4 m deep and the media are 3 to 6 mm in diameter. The media-specific surface area ranges from 500 to 2000 m^2/m^3. The contact time 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 suspended solids. 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 m prior to backwash. In existing installations, the filters are constructed above grade. The combination of the tall structure (6 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 a significant, but intermittent, energy demand.

Process air is required in BAF cells that are removing carbonaceous organic matter (biochemical oxygen demand or BOD) and are nitrifying ammonia-nitrogen. The process aeration system consists of coarse to medium bubble diffusers on a stainless steel piping grid. Because of the difficulty in accessing 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.

5.13.1.2. Configurations

BAFs can operate in different process configurations, depending on the facilities, 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 and suspended solids loadings. Following primary sedimentation, BAF cells can be operated for carbonaceous BOD removal or, under lower loading rates (less than 1.5 kg BOD/m^{3}d), for both carbonaceous BOD and ammonia-nitrogen removal. A cell can operate in a nitrification mode following an activated sludge system or another BAF cell removing carbonaceous BOD. A denitrification BAF process can follow either an activated sludge or BAF system that is nitrifying.

5.13.1.3. 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 suspended solids 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:
1. Warm water temperature;
2. Adequate aeration and good air distribution, and
3. Low carbonaceous BOD and suspended solids loading.

Denitrification usually requires methanol addition, and water velocities must be greater than 10 m/hr.

5.13.2. Moving Bed Biofilm Reactors

The Norwegian company Kaldnes Miløteknologi (KMT) developed the patented MBBR process. The basic concept of the MBBR is to have continuously operating, non-cloggable 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. However, 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 places 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 "dead" space or unused space in the reactor. Organic loading rates for these reactors are typically in the order of 3.5 – 7.0 g BOD/m^2 of media surface area/d for BOD removal and less than 3.5 g BOD/m^2 of media surface area/d for nitrification.

5.13.3. 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.

5.13.3.1. Configuration

Membrane bioreactors can be configured in a number of 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 airflow 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.

5.13.3.2. 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. Typical effluent quality is presented in Table 5.21. 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 for membrane processes as 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 levels of biomass ensure that in the anoxic zone, at all times there are enough denitrifiers to efficiently convert the nitrates into nitrogen gas.

Table 5.21
Membrane Bioreactor Effluent Quality

Parameters	Secondary Treatment	Tertiary Treatment	Membrane Bioreactor
BOD (mg/L)	10-12	< 5	< 2
TSS (mg/L)	10-15	< 1	< 1
NH_3 (mg/L)	1-10	1-10	< 0.3
Total P (mg/L)	> 1	0.1-0.5	< 0.1
Total Coliforms (MPN/100 mL)	> 1000	> 1000	< 100
Fecal Coliforms (MPN/100 mL)	> 100	> 100	< 10
Sludge Yields (kg/kg BOD_5 removed)	0.3-0.6	0.3-0.6	0.1-0.3

5.13.4. Small Treatment Plants

Small treatment plants generally utilize the biological treatment process and are installed as a single "package" unit. The package treatment plant is usually purchased either complete with steel tankage or with the plant equipment ready for installation in "poured in place" concrete tankage.

The process design shall be in accordance with the guidelines presented in the previous sections.

Simplicity of operation and durability are essential in the design of a small treatment plant, which is quite often in a remote location and frequently only receives minimal operator attention.

It is essential that the treatment plant supplier or his representative provide complete operator training during the plant start-up period.

A comprehensive operating manual shall be provided. The manual shall include such items as follows:

1. Process operation, including flow diagrams and comprehensive troubleshooting and remedial action list;
2. Maintenance schedule, including frequency of attention required, e.g. daily, weekly, etc.;
3. Comprehensive equipment information, including drawings, serial numbers, part numbers, etc.; and
4. Safety precautions and action required in the event of an emergency.

5.13.4.1. Flows

When insufficient data is available, the values listed in Table 5.1, may be used in computing the design flow.

5.14. Nutrient Removal and Tertiary Treatment

5.14.1. Phosphorous Removal

5.14.1.1. Applicability

The following factors should be considered when determining the need for phosphorus control at municipal wastewater treatment facilities:

1. The present and future phosphorus loadings from the existing municipal wastewater treatment facility to the receiving water;

2. The CCME Ambient Water Quality Guidelines

3. The existing phosphorus levels in the receiving water and the effects of these levels on the rate of eutrophication along the entire length of receiving waters;

4. The predicted response of the receiving water to increased phosphorus loadings;

5. The existing and desired water quality of the receiving water along its entire length;

6. The existing and projected uses of the receiving water; and

7. Consideration of the best practicable technology available to control phosphorus discharges.

5.14.1.2. Phosphorus Removal Criteria

A municipal wastewater treatment facility shall be required to control the discharge of phosphorus if the following conditions exist:

1. Eutrophication of the receiving water environment is either occurring or may occur at a rate which may affect the existing and potential uses of the water environment; or

2. The municipal wastewater effluent discharge is contributing or may contribute significantly to the rate of receiving water eutrophication.

5.14.1.3. Method of Removal

Acceptable methods for phosphorus removal shall include chemical precipitation, high rate filtration or biological processes.

5.14.1.4. Design Basis

5.14.1.4.1. 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 facility should be based on such factors as influent wastewater characteristics, effluent requirements, and anticipated treatment efficiency.

5.14.1.4.2. System Flexibility

Systems shall be designed with sufficient flexibility to allow for several operational adjustments in chemical feed location, chemical feed rates, and for feeding alternate chemical compounds.

5.14.1.5. Effluent Requirements

If phosphorus control is required, the maximum acceptable concentration of final effluent phosphorus and/or the maximum acceptable mass loading to the receiving stream shall be established on a site-specific basis.

5.14.1.6. Process Requirements

5.14.1.6.1. Dosage

Typical chemical dosage requirements of various chemicals required for phosphorus removal are outlined in Table 5.22.

Dosages will vary with the phosphorus concentration in the effluent. The required chemical dosage shall 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.

Table 5.22
Typical Chemical Dosage Requirements for Phosphorus Removal

Type of Treatment	Addition Point	Dosage Rate (mg/L)		
Plant		**Chemical**	**Range**	**Average**
Mechanical:				
Primary	Raw Sewage	Alum Ferric Chloride Lime	100 6 – 30 167 - 200	100 16 185
Secondary	Raw Sewage	Lime Alum Ferric Chloride	- 40 – 100 -	- 70 -
	Secondary Section	Lime Alum Ferric Chloride	- 30 – 150 2 - 30	- 65 11
Lagoons:				
Seasonal Retention Lagoons	Batch Dosage to Cells	Alum Ferric Chloride Lime	100 – 210 17 – 22 250 - 350	163 20 300
Continuous Discharge Lagoons	Raw Sewage	Alum Ferric Chloride Lime	225 20 400	225 20 400

5.14.1.6.2. 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.

5.14.1.6.3. Chemical Feed System

In designing the chemical feed system for phosphorus removal, the following points should be considered:

1. The need to select chemical feed pumps, storage tanks and piping suitable for use with the chosen chemicals(s);

2. Selection of chemical feed equipment with the required range in capacity;

3. The need for a standby chemical feed pump;

4. Provision of flow pacing for chemical pumps proportional to sewage flow rates;

5. Flexibility by providing a number of chemical application points;

6. The need for protection of storage and piping from the effect of low temperatures;

7. Selection of the proper chemical storage volume;

8. The need for ventilation in chemical handling rooms; and

9. Provision for containment of any chemical spills.

5.14.1.6.4. Chemical Feed Points

Selection of chemical feed points shall 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.

5.14.1.6.5. 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.

5.14.1.6.6. 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 in order to obtain optimum floc growth, control deposition of solids, and prevent floc destruction.

5.14.1.6.7. Liquid - Solids Separation

The velocity through pipes or conduits from flocculation basins to settling basins should not exceed 0.5 m/s in order to minimize floc destruction. Entrance works to settling basins should also be designed to minimize floc shear.

Settling basin design shall be in accordance with criteria outlined in Section 5.6.

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.

5.14.1.6.8. **Filtration**
Effluent filtration shall be considered where effluent phosphorus concentrations of less than 1 mg/L must be achieved.

5.14.1.7. Feed Systems

5.14.1.7.1. **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 so as to minimize the length of slurry conduits. All slurry conduits shall be accessible for cleaning.

5.14.1.7.2. Liquid Chemical Feed System
Liquid chemical feed pumps should be of the positive displacement type with variable feed rate. Pumps shall 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 shall be provided on the chemical feed pump suction lines.

An air break or anti-siphon device shall be provided where the chemical solution stream discharges to the transport water stream to prevent an induction effect resulting in overfeed.

Consideration shall be given to providing pacing equipment to optimize chemical feed rates.

5.14.1.7.3. Dry Chemical Feed System
Each dry chemical feeder shall be equipped with a dissolver, which is capable of providing 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 shall be provided with an educator 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.

5.14.1.8. Storage Facilities

5.14.1.8.1. **Size**
Storage facilities shall be sufficient to insure 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-days supply should be provided.

***5.14.1.8.2.* Location**

The liquid chemical storage tanks and tank fill connections shall be located within a containment structure having a capacity exceeding the total volume of all storage vessels. Valves on discharge lines shall be located adjacent to the storage tank and within the containment structure.

Auxiliary facilities, including pumps and controls, within the containment area shall be located above the highest anticipated liquid level. Containment areas shall be sloped to a sump area and shall not contain floor drains.

Bag storage should be located near the solution make-up point to avoid unnecessary transportation and housekeeping problems.

***5.14.1.8.3.* 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 shall have reasonable access provided to facilitate cleaning.

5.14.1.9. Other Requirements

***5.14.1.9.1.* Materials**

All chemical feed equipment and storage facilities shall be constructed of materials resistant to chemical attack by all chemicals normally used for phosphorus treatment.

5.14.1.9.2. Temperature, Humidity and Dust Control

Precautions shall 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.

***5.14.1.9.3.* Cleaning**

Consideration shall be given to the accessibility of piping. Piping should be installed with plugged wyes, tees or crosses at changes in direction to facilitate cleaning.

5.14.1.9.4. Drains and Drawoff

Above-bottom drawoff from chemical storage or feed tanks shall be provided to avoid withdrawal of settled solids into the feed system. A bottom drain shall also be installed for periodic removal of accumulated settled solids. Provisions shall be made in the fill lines to prevent back siphonage of chemical tank contents.

5.14.1.10. Hazardous Chemical Handling
The requirements of Occupational Health and Safety, and Hazardous Chemical Handling shall be met.

5.14.1.11. Sludge Handling and Dewatering
Consideration shall be given to the type and additional capacity of the sludge handling facilities needed when chemicals are added.

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, dewater ability, final disposal, and operating cost.

5.14.2. Ammonia Removal

5.14.2.1. Breakpoint Chlorination

5.14.2.1.1. Applicability
The breakpoint chlorination process is best suited for removing relatively small quantities of ammonia, less than 5 mg/L NH_3-N, and in situations whose low residuals of ammonia or total nitrogen are required.

5.14.2.1.2. Design Considerations

5.14.2.1.2.1. 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 backmixed reactors. A minimum contact time of 10 minutes is recommended.

5.14.2.1.2.2. 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 pretreatment increases, the required amount of chlorine decreases and approaches the theoretical amount required to oxidize ammonia to nitrogen (7.6 mg/L Cl_2:1 mg/L NH_3-N). Table 5.23 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.

5.14.2.1.2.3. 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.

Table 5.23
Quantities of Chlorine Required for Three Wastewater Sources

Wastewater Source	Chlorine: NH_3 – N Ratio to Reach Breakpoint	
	Experience	Recommended Design Capability
Raw	10:1	13:1
Secondary Effluent	9:1	12:1
Lime Settled and Filtered Secondary Effluent	8:1	10:1

5.14.2.1.2.4. 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 shall also be examined when sizing the equipment.

5.14.2.1.2.5. pH Adjustment

Except for wastewaters having a high alkalinity or treatment systems employing lime coagulation prior to chlorination, provisions shall 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.

5.14.2.2. Air Stripping

5.14.2.2.1. 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, and 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 insure 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.

5.14.2.2.2. Design Considerations

5.14.2.2.2.1. 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 opening spacing is used where lower levels of ammonia removal are acceptable. Because of 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 to 7.5 m should be used to minimize power costs.

5.14.2.2.2.2. Hydraulic Loadings

Allowable hydraulic loading is dependent on the type and spacing of the individual splash bars. Although hydraulic loading rates used in ammonia stripping towers should range from 0.7 to 2.0 L/m^2s removal efficiency is significantly decreased at loadings in excess of 1.3 L/m^2s. The hydraulic loading rate should be such that a water droplet is formed at each individual splash bar as the liquid passes through the tower.

5.14.2.2.2.3. Air Requirements

Air requirements vary from 2200 to 3800 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.

5.14.2.2.2.4. 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.

5.14.2.2.2.5. General Construction Features

The stripping tower may be either of the counter-current (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 shall 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.

5.14.2.2.2.6. Process Control

During periods of tower operation when temperature, air and wastewater flow rates, and scale formation are under control, the major process requirement necessary to insure 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).

5.14.3. Biological Nutrient Removal

5.14.3.1. 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."

5.14.3.1.1. Mainstream Phosphorus Removal (A/O Process)

The proprietary A/O process is a single sludge suspended-growth system that combines anaerobic and aerobic sections 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 waste activated sludge. The concentration of phosphorus in the effluent is dependent mainly on the ratio of BOD to phosphorus of the wastewater treated.

5.14.3.1.2. Sidestream Phosphorus Removal (PhoStrip Process)

In the proprietary PhoStrip Process a portion of the return activated sludge 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.

5.14.3.1.3. Design Criteria

Table 5.24 provides various design criteria with regards to biological phosphorous removal processes.

Table 5.24
Design Criteria for Biological Phosphorus Removal

Design Parameter	Treatment Process		
	A/O	**PhoStrip**	SBR
Food/Microorganism Ratio (kg BOD_5/kg MLVSS.d)	0.2 – 0.7	0.1 – 0.5	0.15 – 0.5
Solids Retention Time (day)	2 - 25	10 – 30	
MLSS (mg/L)	2000 – 4000	600 - 5000	2000 – 3000
Hydraulic Retention Time (hours)			
Anaerobic Zone	0.5 – 1.5	8 – 12	1.8 – 3
Aerobic Zone	1 - 3	4 - 10	1.0 – 4
Return Activated Sludge (% of Influent Flowrate)	25 - 40	20 - 50	N/A
Stripper Underflow (% of Influent Flowrate)	N/A	10 - 20	N/A

5.14.3.2. 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.

5.14.3.2.1. 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.

Table 5.25 provides design criteria for various nitrification processes.

Table 5.25
Design Criteria for Nitrification

Design Parameter	Single Stage	Separate Storage
Food/Microorganism Ratio (kg Bod5/kg MLVSS.day)	0.12 – 0.25	0.05 – 0.2
Solids Retention Time (day)	8 – 20	15 – 100
MLSS (mg/L)	1500 – 3500	1500 – 3500
Hydrauic Retention Time (hours)	6 – 15	3 – 6
Return Activated Sludge (% of Influent Flowrate)	50 - 150	50 - 200

5.14.3.2.2. Combined Nitrification/Denitrification

The removal of nitrogen by biological nitrification/denitrification is a two-step process. In the first step, ammonia is converted aerobically to nitrate ($NO3^-$) (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.

5.14.3.2.2.1. 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. Because 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.

5.14.3.2.2.2. 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. Because the system has only one anoxic zone, nitrogen removals are lower than those of the Bardenpho process.

5.14.3.3. Combined Biological Nitrogen and Phosphorus Removal

A number of 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.

5.14.3.3.1. A^2/O Process

The proprietary A^2/O process provides an anoxic zone for denitrification with a detention period of approximately one hour. The anoxic zone is deficient in dissolved oxygen, but chemically bound oxygen ion the form of nitrate or nitrite is introduced by recycling nitrified mixed liquor from the aerobic section. Effluent phosphorus concentrations of less than 2 mg/L can be expected without effluent filtration; with effluent filtration, effluent phosphorus concentrations may be less than 1.5 mg/L.

5.14.3.3.2. Bardenpho 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 five-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.

5.14.3.3.3. **UCT Process**

The UCT process eliminates return activated sludge 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.

5.14.3.3.4. Design Criteria

Table 5.26 provides design criteria for various combined biological nitrogen and phosphorous removal processes.

Table 5.26
Design Criteria for Combined Biological Nitrogen and Phosphorus Removal

Design Parameter	Treatment Process			
	A^2/O	**Bardenpho (5 Stage)**	**UCT**	**SBR**
Food/Microorganism Ratio (kg BOD5/kg MLVSS.day)	0.15 – 0.25	0.1 – 0.2	0.1 – 0.2	0.1
Solids Retention Time (day)	4 - 27	10 - 40	10 - 30	-
MLSS (mg/L)	3000 - 5000	2000 - 4000	2000 - 4000	600 - 5000
Hydraulic Retention Time (hrs)				Batch Times
Anaerobic Zone	0.5 – 1.5	1 – 2	1 – 2	0 – 3
Anoxic Zone – 1	0.5 – 1.0	2 – 4	2 – 4	0 – 1.6
Aerobic Zone – 1	3.5 – 6.0	4 – 12	4 – 12	0.5 – 1
Anoxic Zone – 2		2 – 4	2 – 4	0 – 0.3
Aerobic Zone – 2		0.5 – 1		0 – 0.3
Settle/Decant				1.5 – 2
Total	4.5 – 8.5	9.5 - 23	9 - 22	4 – 9
Return Activated Sludge % of Influent Flowrate)	20 - 50	50 - 100	50 - 100	-
Internal Recycle (% of Influent Flowrate)	100 - 300	400	100 - 600	-

5.14.3.4. Sequencing Batch Reactor (SBR)

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 of 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.

5.14.4. Effluent Filtration

5.14.4.1. Applicability

Effluent filtration is generally necessary when effluent quality better than 15 mg/L BOD_5, 15 mg/L suspended solids and 1.0 mg/L of phosphorus is required.

Where effluent suspended solids 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.

5.14.4.2. Design Considerations

Factors to consider when choosing between the different filtration systems which are available, include the following:

1. The installed capital and expected operating and maintenance costs;

2. The energy requirements of the systems (head requirements);

3. The media types and sizes and expected solids capacities and treatment efficiencies of the system; and

4. The backwashing systems, including type, backwash rate, backwash volume, effect on sewage 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.

5.14.4.3. Location of Filter System

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).

5.14.4.4. Number of Units

Total filter area shall be provided in 2 or more units, and the filtration rate shall be calculated on the total available filter area with one unit out of service.

5.14.4.5. Filter Types

Filters may be of the gravity type or pressure type. Pressure filters shall 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.

5.14.4.6. Filtration Rates

5.14.4.6.1. Hydraulic Loading Rate

Filtration rates at peak hourly sewage flow rates, including backwash flows, should not exceed 2.1 L/m^{2}s for shallow bed single media systems (if raw sewage 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 sewage flow rates, including backwash flows, should not exceed 3.3 L/ m^{2}s for deep bed filters (if raw sewage flow equalization is provided, lower peak filtration rates should be used in order to avoid undersizing of the filter). The manufacturer's recommended maximum filtration rate should, however, not be exceeded.

5.14.4.6.2. Organic Loading Rate

Peak solids loading rate should not exceed 50 mg/m^{2}s for shallow bed filters and 80 mg/m^{2}s for deep bed filters (if raw sewage flow equalization is provided, lower peak solids loading rates should be used in order to avoid undersizing of the filter).

5.14.4.7. Backwash

Pumps for backwashing filter units shall 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 shall 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 flow rate of the plant, a backwash holding tank should be provided to equalize the flow of backwash water to the plant.

5.14.4.7.1. Backwash Rate

The backwash rate shall be adequate to fluidize and expand each media layer a minimum of 20% based on the media selected. The backwash system shall be capable of providing a variable backwash rate having a maximum of at least 14 L/m^{2}s and a minimum backwash period of 10 minutes.

5.14.4.8. Filter Media

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.

Table 5.27 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 5.27
Media Depths and Sizes

(Minimum Depth) (Effective Size)			
	Single Media	**Multi-Media** (2)	(3)
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 06-0.8 mm
Garnet or Similar Material	-	-	5 cm 0.3-0.6 mm
Uniformity Coefficient shall be 1.7 or less			

5.14.4.9. Filter Appurtenances

The filters shall be equipped with wash water troughs, surface wash or air scouring equipment, 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, and filter influent and effluent sampling points. If automatic controls are provided, there shall be a manual override for operating equipment, including each individual valve essential to the filter operation. The underdrain system shall be designed for uniform distribution of backwash water (and air, if provided) without danger of clogging from solids in the backwash water.

Provision shall 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 shall be provided.

5.14.4.10. Reliability

Each filter unit shall 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 shall depend on expected extreme climatic conditions at the treatment plant site. As a minimum, all controls shall be enclosed. The structure housing filter controls and equipment shall be provided with adequate heating and ventilation equipment to minimize problems with excess humidity.

5.14.4.11. 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 flow rate to the treatment units. The hydraulic and organic load from waste backwash water shall be considered in the overall design of the treatment plant.

Surge tanks shall 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 shall be provided with the largest unit out of service.

5.14.4.12. Backwash Water Storage

Total backwash water storage capacity provided in an effluent clearwell or other unit shall equal or exceed the volume required for two complete backwash cycles.

5.14.4.13. 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 shall be provided. Such equipment will be reviewed on a case-by case basis at the discretion of the DOEC.

5.14.5. Microscreening

5.14.5.1. Applicability

Microscreening units may be used following a biological treatment process for the removal of residual suspended solids. 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.

5.14.5.2. Design Considerations

Pilot plant testing on existing secondary effluent is encouraged. Where pilot studies so indicate, where microscreens follow trickling filters or lagoons, or where effluent suspended solids requirements are less than 10 mg/L, a pre-treatment process such as chemical coagulation and sedimentation shall be provided. Care should be taken in the selection of pumping equipment ahead of microscreens to minimize shearing of floc particles. The process design shall include flow equalization facilities to moderate microscreen influent quality and quantity.

5.14.5.3. Screen Material

The micro-fabric shall 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 microns. The use of pilot plant testing for aperture size selection is recommended.

5.14.5.4. Screening Rate

The screening rate shall be selected to be compatible with available pilot plant test results and selected screen aperture size, but shall not exceed 3.4 L/m^2s of effective screen area based on the maximum hydraulic flow rate applied to the units. The effective screen area shall be considered as the submerged screen surface area less the area of screen blocked by structural supports and fasteners. The screening rate shall be that applied to the units with one unit out of service.

5.14.5.5. Backwash

All waste backwash water generated by the microscreening operation shall be recycled for treatment. The backwash volume and pressure shall 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, shall be provided. Backwash water shall 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 shall be controlled such that the rate does not exceed 15% of the design average daily flow rate to the treatment plant. The hydraulic and organic load from waste backwash water shall be considered in the overall design of the treatment plant.

Where waste backwash is returned for treatment by pumping, adequate pumping capacity shall be provided with the largest unit out of service. Provisions should be made for measuring backwash flow.

5.14.5.6. Appurtenances

Each microscreen unit shall be provided with automatic drum 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 shall be provided. The use of chlorine should be restricted to those installations where the screen material is not subject to damage by the chlorine.

5.14.5.7. Reliability

A minimum of two microscreen units shall be provided, each unit being capable of independent operation. A supply of critical spare parts shall be provided and maintained. All units and controls shall be enclosed in a heated and ventilated structure with adequate working space to provide for ease of maintenance.

5.14.6. Activated Carbon Adsorption

5.14.6.1. 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.

5.14.6.2. Design Considerations

The usefulness and efficiency of carbon adsorption for municipal 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 suspended solids, oxygen demand, other organics such as methylene blue active substance (MBAS) or phenol, and dissolved oxygen. 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:

1. Attraction of carbon for solute;
2. Attraction of carbon for solvent;
3. Solubilizing power of solvent or solute;
4. Association;
5. Ionization;
6. Effect of solvent on orientation at interface;
7. Competition for interface in presence of multiple solutes;
8. Co-adsorption;
9. Molecular size of molecules in the system;
10. Pore size distribution in carbon;
11. Surface area of carbon; and
12. 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.

5.14.6.3. Unit Sizing

5.14.6.3.1. **Contact Time**

The contact time shall be calculated on the basis of the volume of the column occupied by the activated carbon. Generally, carbon contact times of 15 to 35 minutes are used depending on the application, the wastewater characteristics, and the desired effluent quality. For tertiary treatment applications, carbon contact times 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 contact times of 20 to 35 minutes should be used, with a contact time of 30 minutes being typical.

5.14.6.3.2. Hydraulic Loading Rate

Hydraulic loading rates of 2.5 to 7.0 L/m^{2}s of cross section of the bed shall be used for upflow carbon columns. For downflow carbon columns, hydraulic loading rates of 2.0 to 3.3 L/ m^{2}s are used. Actual operating pressure seldom rises above 7 kN/m^2 for each 0.3 m of bed depth.

5.14.6.3.3. **Depth of Bed**

The depth of bed will vary considerably, depending primarily on carbon contact time, and may be from 3 to 12 m. A minimum carbon depth of 3 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.

5.14.6.3.4. 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 insure an adequate carbon contact time to maintain effluent quality while one column is off line during removal of spent carbon for regeneration or for maintenance.

5.14.6.4. Backwashing
The rate and frequency of backwash is dependent on hydraulic loading, the nature and concentration of suspended solids 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 flow rates for granular carbons of 8 x 12 or 12 x 30 mesh are 8 to 14 L/m^2s.

5.14.6.5. Valve and Pipe Requirements
Upflow units shall be piped to operate either as upflow or downflow units as well as being capable of being backwashed. Downflow units shall 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.

5.14.6.6. Instrumentation
The individual carbon columns should be equipped with flow and headloss measuring devices.

5.14.6.7. Hydrogen Sulphide Control
Methods that can be incorporated into the plant design to cope with hydrogen sulphide production include:

1. Providing upstream biological treatment to satisfy as much of the biological oxygen demand as possible prior to carbon treatment;

2. Reducing detention time in the carbon columns based on dissolved oxygen concentrations of the effluent;

3. Backwashing the columns at more frequent intervals;

4. Chlorinating carbon column influent; and

5. In upflow expanded beds, the introducing of an oxygen source, such as air or hydrogen peroxide, to keep the columns aerobic.

5.14.6.8. 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 insure that gravel or stone supporting media used in downflow contactors does not enter the carbon transport system.

Activated carbon shall 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 owner preference, column control capabilities, capital and maintenance costs, and pumping head requirements.

Carbon slurry piping systems shall 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.

5.14.6.9. Carbon Regeneration

5.14.6.9.1. 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 5.28.

Table 5.28
Typical Carbon Dosages for Different Column Wastewater Influents

Pretreatment	Typical Carbon Dosage Required per m^2 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.

5.14.6.9.2. Carbon Dewatering

Dewatering of the spent carbon slurry prior to thermal regeneration may be accomplished in spent carbon drain bins. The drainage bins shall be equipped with screens to allow the transport of water to flow from the carbon. Two drain bins shall 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.

5.14.6.9.3. 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 kg of steam per kg of carbon regenerated. Fuel requirements for the carbon regeneration furnace are 7000 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 shall 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 shall be designed as an integral part of the furnace and include a scrubber for removing carbon fines and an afterburner for controlling odours.

5.14.7. Constructed Wetlands
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, 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. Because 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.

5.14.7.1. Types
Wastewater treatment systems using constructed wetlands have been categorized as either free water surface (FWS) or subsurface flow (SFS) types:

1. **Free Water Surface Wetlands (FWS) -** 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. FWS wetlands are typically long and narrow to minimize short-circuiting.

2. **Subsurface Flow Wetlands (SFS)** - A SFS 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 and 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.

5.14.7.2. Site Evaluation

Site characteristics that must be considered in wetland system design include topography, soil characteristics, existing land use, flood hazard, and climate, and are detailed as follows:

1. **Topography** - Level to slightly sloping, uniform topography is preferred for wetland sites because free water systems (FWS) are generally designed with level basins or channels, and subsurface flow systems (SFS) 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%.

2. **Soil** - Sites with slowly permeable (<0.5 cm/h) 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. Compacting during construction may purposely reduce permeability of native soils. Sited with rapidly permeable 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 - 0.6 m to allow sufficient distance for treatment of any percolate entering the groundwater.

3. **Flood Hazard** - Wetland sites should be located outside of flood plains, or protection from flooding should be provided.

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.

5. **Climate** - The use of wetland systems in cold climates is possible. Because the principle treatment systems are biological, treatment performance is strongly temperature sensitive. Storage will be required where treatment objectives cannot be met due to low temperatures.

5.14.7.3. Preapplication Treatment

Artificial wetlands may be designed to accept wastewater with minimal (coarse screening and comminution) pretreatment. However, the level of pretreatment 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 pretreatment step to ensure continued satisfactory phosphorus removal within the marsh.

5.14.7.4. Vegetation Selection and 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 SFS systems. Cattails tend to dominate in water depths over 0.15 m. Bulrushes grow well at depths of 0.05 - 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 shallower 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 SFS systems because the depth of rhizome penetration allows for the use of deeper basins.

Harvesting of wetland vegetation is generally not required, especially for SFS systems. However dry grasses in FWS systems are burned off periodically to maintain free-flow conditions and to prevent channelling of the flow. Removal of the plant biomass for the purpose of nutrient removal is normally not practical.

5.14.7.5. Design Parameters

5.14.7.5.1. Detention Time

Free Water Surface Wetlands (FWS) - The relationship between BOD removal and detention times for FWS is represented by the equation:

$$Ce = Co^{(-k_T t)}$$

where: Ce = effluent BOD (mg/L);
Co = influent BOD (mg/L);
kT = temperature dependent rate constant;
$d^{-1} = k_{20} \times 1.06^{(T-20)}$;
$k_{20} = 0.678d^{-1}$;
T = average monthly water temperature (^{o} C);
t = average detention time (d)

$$t = \frac{LWnd}{Q}$$

where: L = basin length (m);
W = basic width (m);
n = fraction of cross sectional area not used by plants (0.65-0.75);
d = depth of basin (m); and
Q = average flowrate through system [$(Q_{in} + Q_{out}) / 2$], (m^3/d)

$$Q = \frac{Q_{in} + Q_{out}}{2}$$

Subsurface Flow Systems (SFS) - The relationship between BOD removal and detention times for SFS is represented by the equation:

$$Ce = Co^{(-K\tau t')}$$

where: Ce = effluent BOD (mg/L);
Co = influent BOD (mg/L);
K_T = temperature dependent rate constant;
$d^{-1} = K_{20} \times 1.06^{(T-20)}$
T = average monthly water temperature (^{o} C);
$K_{20} = 1.104\ d^{-1}$

$$t' = \frac{LW\alpha d}{Q}$$

where: L = basin length (m);
W = basin width (m);
α = porosity of basin medium = 0.35 (gravelly sand); 0.30 (coarse sand); 0.28 (medium sand)
d = depth of basin (m); and
Q = average flowrate through system (m^3/d)

$$Q = \frac{Q_{in} + Q_{out}}{2}$$

5.14.7.5.2. 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. Water depths should range from 0.1 - 0.5 m.

The design depth of SFS 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. The media depth may range from 0.3 - 0.75 m.

5.14.7.5.3. Aspect Ratio

The aspect ration for FWS wetlands is important to the performance for removal of BOD, TSS, NH_3, and total nitrogen. Length to width ratios of 4:1 to 6:1 are needed to achieve expected performances and avoid short-circuiting of wastewater through the wetland. For large systems, an aspect ratio of 2:1 is the minimum recommended.

For SFS wetlands the bed width is determined by the hydraulic flowrate. The length of the bed is determined by the needed detention time for pollutant removal.

Therefore SFS wetlands may have aspect ratios less than or greater than 1:1 depending on the treatment goal.

5.14.7.5.4. Loading Rates

Table 5.29 summarizes the hydraulic, BOD, and SS loading rates for BOD removal in both FWS and SFS systems.

Table 5.29
Loading Rates for Constructed Wetlands

Wetland Type	Hydraulic Loading Rate m^3/ha.d	Maximum BOD Loading Rate kg/ha.d	Maximum SS Loading Rate at Inlet kg/m^2d
Free Water Surface	150 - 500	65	Not applicable
Subsurface Flow System	Not Applicable	65	0.08

6.14.7.5.5. Nutrient Removal

Free Water Surface Wetlands (FWS) - Detention times for nutrient removal need to be longer than the 5 - 10 days required for BOD and SS. For ammonia or total nitrogen removal, both minimum temperature and detention time are important. Detention times for significant nitrogen removal should be 8 - 14 days or more. Nitrogen removal and nitrification will be reduced when water temperatures fall below 10ºC and should not be expected when water temperatures fall below 4ºC.

Plant uptake of phosphorus is rapid, and following plant death, phosphorus may be quickly recycled to the water column or deposited in the sediments. The only major sink for phosphorus in most wetlands is in the soil. Significant phosphorus removal requires long detention times (15 - 20 days) and low phosphorus loading rates (< 0.3 kg/ha·d).

Subsurface Flow Wetlands (SFS) - Both detention time and oxygen transfer can limit nitrification and subsequent nitrogen removal in SFS wetlands. Because nitrification of 20 mg/L of ammonia will require 100 mg/L of oxygen, oxygen transfer is critical to nitrification in SFS wetlands. Plant roots can generate a portion of this demand for oxygen in the subsurface, however, direct oxygen transfer from the atmosphere may be required to achieve effective nitrification. The detention time and temperature limits for FWS apply to SFS wetlands.

5.14.7.6. Vector Control
FWS systems provide ideal breeding habitat for mosquitoes. Plans for biological control of mosquitoes through the use of mosquito fish and sparrows plus application of chemical control agents as necessary must 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 SFS 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.

5.14.7.7. Vegetation Harvesting
Harvesting of the emergent vegetation is only required to maintain hydraulic capacity, promote active growth, and avoid mosquito growth. Harvesting for nutrient removal is not practical and is not recommended.

5.14.7.8. Monitoring
Monitoring is necessary to maintain loadings within design limits. A routine monitoring program should be established for the following parameters:
1. Wastewater application rates (m^3/d);
2. Discharge flow rates (m^3/d);
3. Wastewater quality, including BOD_5 and COD, suspended solids, total dissolved solids, total nitrogen, total phosphorous, pH and sodium adsorption ratio; and
4. Discharge water quality according to the analyses summarized in item 3 above.

5.14.8. Floating Aquatic Plant Treatment Systems
Aquatic treatment systems consist of one or more shallow ponds in which one or more species of water tolerant vascular plants such as water hyacinths or duckweed are grown. The shallower depths and the presence of aquatic macrophytes in the place of algae are the major differences between aquatic treatment systems and stabilization ponds. The presence of plants is of great practical significance because the effluent from aquatic systems is of higher quality than the effluent from stabilization pond systems for equivalent or shorter detention times. This is true, particularly when the systems are situated after conventional pond systems, which provide greater than primary treatment.

In aquatic systems, wastewater is treated principally by bacterial metabolism and physical sedimentation, as is the case in conventional trickling filter systems. The aquatic plants themselves bring about very little actual treatment of the wastewater. Their function is to provide components of the aquatic environment that improve the wastewater treatment capability and/or reliability of that environment.

5.14.8.1. Plant Selection

The principal floating aquatic plants used in aquatic treatment systems are water hyacinth, duckweed and pennywort. These plants are described in greater detail in the following discussion.

5.14.8.1.1. Water Hyacinths

Water hyacinth is a perennial, fresh water aquatic vascular plant with rounded, upright, shiny green leaves and spikes of lavender flowers. The petioles of the plant are spongy with many air spaces and contribute to the buoyancy of the hyacinth plant. When grown in wastewater, individual plants range from 0.5 to 1.2 m from the top of the flower to the root tips. The plants spread laterally until the water surface is covered, and then the vertical growth increases. The growth of water hyacinth is influenced by efficiency of the plant to use solar energy, nutrient composition of the water, cultural methods, and environmental factors.

Under normal conditions, loosely packed water hyacinths can cover the water surface at relatively low plant densities, about 10 kg/m^2 wet weight. Plant densities as high as 80 kg/m^2 wet weight can be reached. As in other biological processes, the growth rate of water hyacinths is dependent on temperature. Both air and water temperatures are important in assessing plant vitality.

5.14.8.1.2. Duckweed

Duckweed is a small, green freshwater plant with fronds from one to a few millimetres in width with a short root, usually less than 12 mm in length. Duckweed is the smallest and the simplest of the flowering plants and has one of the fastest reproduction rates. Duckweed grown in wastewater effluent, at 27ºC, doubles in frond numbers, and therefore in area covered, every four days. The plant is made up of essentially metabolically active cells with very little structural fibre.

Small floating plants, particularly duckweed, are sensitive to wind and may be blown in drifts to the leeward side of the pond unless baffles are used. Redistribution of the plants requires manual labour. If drifts are not redistributed, decreased treatment efficiency may result due to incomplete coverage of the pond surface. Odours have also developed where accumulated plants are allowed to remain and undergo anaerobic decomposition.

5.14.8.1.3. Pennywort

Pennywort is generally a rooted plant. However, under high-nutrient conditions, it may form hydroponic rafts that extend across water bodies. Pennywort tends to intertwine and grows horizontally; at high densities, the plants tend to grow vertically. Unlike water hyacinth, the photosynthetic leaf area of pennywort is small, and, at dense plant stands, yields are significantly reduced as a result of self-shading. Pennywort exhibits mean growth rates greater than 0.010 kg/m^2d in warm climates. Although rates of nitrogen and phosphorous uptake by water hyacinth drop sharply during the winter, nutrient uptake by pennywort is approximately the same during both warm and cool seasons. Pennywort is a cool season plant that can be integrated into water hyacinth/water lettuce biomass production systems.

5.14.8.2. Types of Systems

The principal types of floating aquatic plant treatment systems used for wastewater treatment are those employing water hyacinth and duckweed.

5.14.8.2.1. Water Hyacinth Systems

Water hyacinth systems represent the majority of aquatic plant systems that have been constructed. Three types of hyacinth systems can be described based on the level of dissolved oxygen and the method of aerating the pond:

1. Aerobic non-aerated;
2. Aerobic aerated; and
3. Facultative anaerobic.

A non-aerated aerobic hyacinth system will produce secondary treatment or nutrient (nitrogen) removal depending on the organic-loading rate. This type of system is the most common of the hyacinth systems now in use. The advantages of this type of system include excellent performance with few mosquitoes or odours.

For plant locations in which no mosquitoes or odours can be tolerated, an aerated aerobic hyacinth system is required. The added advantages of such a system are that with aeration, higher organic-loading rates are possible, and reduced land area is required.

The third configuration for a hyacinth system is known as a facultative anaerobic hyacinth system. These systems are operated at very high organic-loading rates.

Odours and increased mosquito populations are the principal disadvantages of this type of system. Facultative anaerobic hyacinth systems are seldom used because of these problems.

5.14.8.2.2. Duckweed Systems

Duckweed and pennywort have been used primarily to improve the effluent quality from facultative lagoons or stabilization ponds by reducing the algae concentration. Conventional lagoon design may be followed for this application, except for the need to control the effects of wind. Without controls, duckweed will be blown to the downwind side of the pond, resulting in exposure of large surface areas and defeating the purpose of the duckweed cover. As noted previously, accumulations of decomposing plants can also result in the production of odours.

Floating baffles can be used to construct cells of limited size to minimize the amount of open surface area exposed to wind action.

5.14.8.3. Climatic Constraints

The water hyacinth systems that are currently used to treat wastewater are located in the warm temperature climates. The optimum water temperature for water hyacinth growth is 21 – 30EC. Air temperatures of –3EC for 12 hours will destroy the leaves and exposure at –5EC for 48 hours will kill the plants. If a water hyacinth system were to be used in a colder climate, it would be necessary to house the system in a greenhouse and maintain the temperature in the optimum range. Duckweed is more cold tolerant than water hyacinths and can be grown practically at temperatures as low as 7EC.

5.14.8.4. Preapplication Treatment

The minimum level of pre-application treatment should be primary treatment, short detention time aerated ponds or the equivalent. Treatment beyond primary depends on the effluent requirements. Use of oxidation ponds or lagoons in which high concentrations of algae are generated should be avoided prior to aquatic treatment because algae removal is inconsistent. When there are effluent limitations on phosphorus, it should be removed in the pre-application treatment step because phosphorus removal in aquatic treatment systems is minimal.

5.14.8.5. Design Parameters

The principal design parameters for aquatic treatment systems include hydraulic detention time, water depth, pond geometry, organic-loading rate, and hydraulic loading rate. Typical design guidelines for water hyacinth and duckweed systems are summarized in Table 5.30 for different levels of pre-application treatment.

Table 5.30
Floating Aquatic Plant System Design Criteria

Item	**Type of Water Hyacinth Treatment System**			**Duckweed Treatment System**
	Secondary Aerobic (non-aerated)	**Secondary Aerobic (aerated)**	**Nutrient Removal Aerobic (non-aerated)**	
Influent Wastewater	Primary Effluent	Primary Effluent	Secondary Effluent	Facultative Pond Effluent
Influent BOD_5 (mg/L)	130 - 180	130 - 180	30	40
BOD_5 Loading (kg/ha.d)	45 - 90	170 - 340	10 - 45	22 – 28
Water Depth (m)	0.5 – 1.0	1.0 – 1.3	0.7 – 1.0	1.3 – 2.0
Detention Time (d)	10 - 36	4 - 8	6 - 18	20 – 25
Hydraulic Loading Rate (m^3/ha.d)	190 - 570	95 - 285	375 - 1500	570 – 860
Water Temperature (ºC)	> 10	> 10	> 10	> 7
Harvest Schedule	Seasonally	Bi-monthly	Bi-monthly	Monthly

5.14.8.6. Pond Configuration

5.14.8.6.1. Water Hyacinth Systems

Typical pond configurations used for water hyacinth systems involve rectangular basins in series similar to stabilization ponds. Recycle and step feed are employed to reduce the concentration of the organic constituent at the plant root zone, improve the transport of wastewater to the root zone, and reduce the formation of odours.

5.14.8.6.2. Duckweed Systems

Duckweed systems should be designed as conventional stabilization ponds except for the need to control the effects of wind. Floating baffles are used to minimize the amount of surface area exposed to direct wind action. Without this control, duckweed will be blown by the wind and treatment efficiencies cannot be achieved.

5.14.8.7. Plant Harvesting and Processing

The need for plant harvesting depends on water quality objectives, the growth rates of the plants, and the effects of predators such as weevils. Harvesting of aquatic plants is needed to maintain a crop with high metabolic uptake of nutrients. Frequent harvesting of hyacinths is practiced to achieve nutrient removal. Significant phosphorus removal is achieved only with frequent harvesting. In areas where weevils pose a threat to healthy hyacinth populations, selective harvesting is often used to keep the plants from being infected. Duckweed harvesting for nutrient removal may be required as often as once per week during warm periods.

Harvested water hyacinth plants are typically dried and landfilled or spread on land and tilled into the soil. Water hyacinth can also be composted readily. However, if the plants are not first partially dried or squeezed, the high moisture content tends to reduce the effectiveness of the compost process and results in the production of a liquid stream that must be disposed of. Ground duckweed can be used as animal feed without air-drying.

5.15. Septic Tank Systems

Septic tanks are principally used for the treatment of household wastes from individual residences. However, if the land availability, soil conditions and other specific criteria are acceptable larger installations can be considered.

Any septic tank system that is part of a municipal system is the responsibility of the DOEC, and all non-municipal systems fall under the jurisdiction of the Department of Government Services.

5.15.1. Definitions

Septic Tank - A water tight, covered receptacle designed and constructed to receive the discharge of sewage from a building sewer; separate solids from the liquid; digest organic matter and store digested solids through a period of detention and allow the clarified liquids to discharge for disposal.

Tile Bed - A system of absorption trenches laid in parallel to distribute the septic tank effluent.

Sand Filter - A bed consisting of a number of lines of perforated pipe or drain tile surrounded by clean, coarse aggregate, containing an intermediate layer of sand as filtering material and provided with a system of underdrains for carrying off the filtered septic tank effluent.

5.15.2. Septic Tank

5.15.2.1. Location

The following are minimum clearance distances, however local conditions may necessitate greater distances:

1. 15 m from any well, lake, stream or pond;
2. 7.5 m from water service line;
3. 15 m from any watermain;
4. 1.5 m from any building; and
5. 3 m from any property boundary.

5.15.2.2. Capacity

The tank shall not be designed to receive weeping tile or roof drainage. Larger tanks are more economical with regard to maintenance, since they do not have to be cleaned as often.

5.15.2.3. Tank Proportions

The length should not be less than 2 or more than 4 times the width.

A minimum of two compartments is required. The volume of the first compartment is to be approximately equal to two-thirds of the total volume.

A typical detail of a septic tank is detailed as Figure 5.3.

5.15.2.4. Detention Time

A primary treatment unit shall have a minimum detention time of 24 hours using average daily flows.

5.15.2.5. Design Flow

When insufficient data is available the values listed in Table 5.1 (page 5-4) may be used in computing thr design flow.

5.15.2.6. Liquid Depth

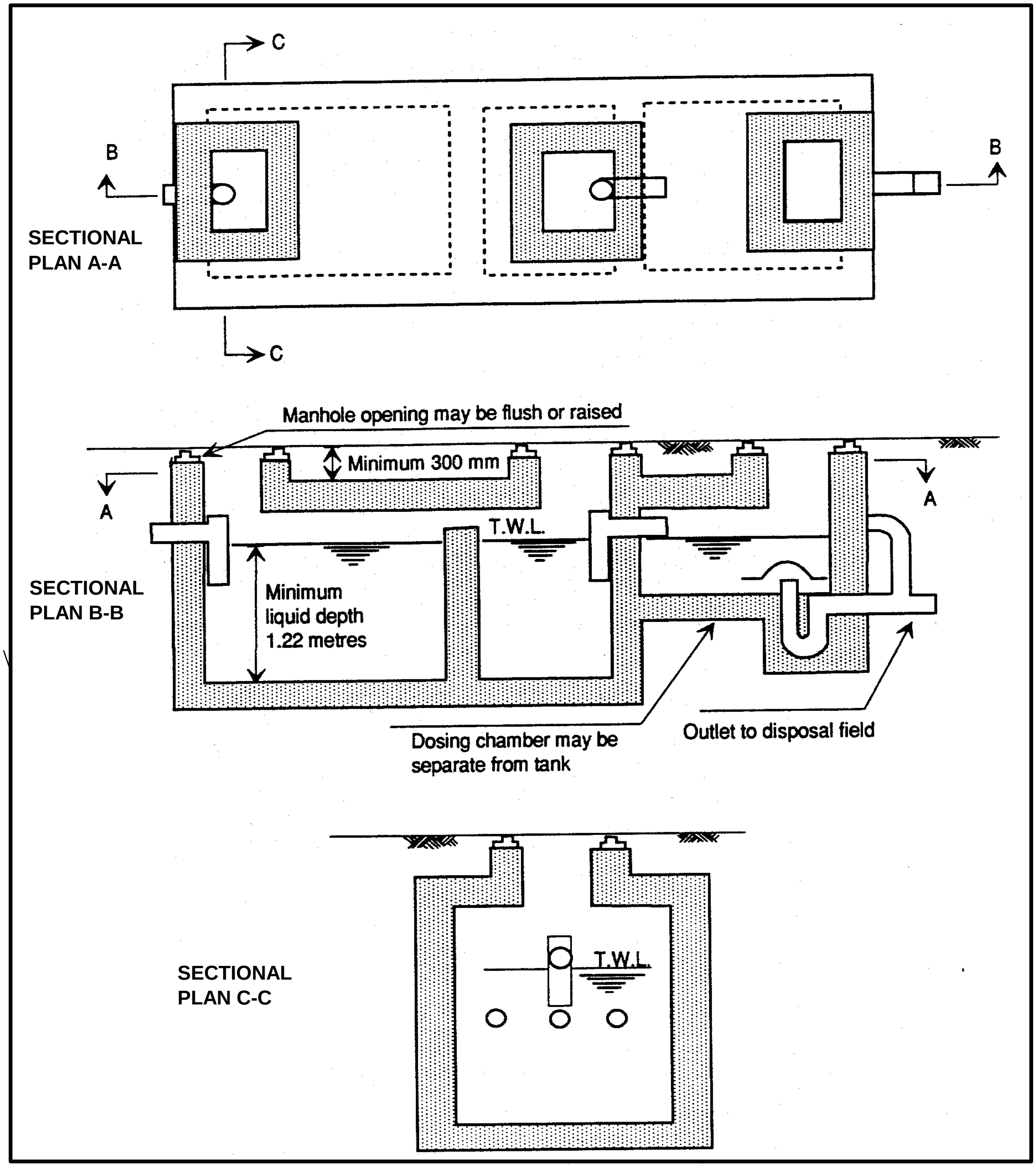
A minimum liquid depth of l.22 m is to be provided.

5.15.2.7. Inlets and Outlets

Centred tee-type inlet and outlet are acceptable. Alternatively baffled weir type outlets can be provided. The invert of the outlet shall be a minimum of 75 mm below the invert of the inlet.

Every tank must have vents in order to prevent sewage flows from draining by vacuum “u”-traps in house plumbing and to also allow the escape of methane and malodorous gases from the tank. In this regard, inlet and outlet tees shall not be capped.

Figure 5.3
Typical Septic Tank Detail



5.15.2.8. Top of Tank

The top of the tank shall be at least 300 mm below the finished grade. There should be at least 25 mm clearance between the underside of the top of the tank and the top of partitions or baffles.

5.15.2.9. Tank Installation

Pre-cast tanks shall be installed according to manufacturer's requirements. Precast/Cast In Place Tanks should be placed on properly designed Type 2 granular bedding in accordance with the Department of Municipal and Provincial Affairs Water, Sewer and Roads Master Specification Section 02223.

5.15.2.10. Manholes

At least one manhole with a minimum dimension of 600 mm shall be provided in each compartment. The manhole is to extend to the ground surface to facilitate inspection and sludge and scum removal. The manholes should be located above the inlet and outlet points.

5.15.2.11. Material of Construction

The tank shall be watertight and constructed of durable material not subject to corrosion, decay, and frost damage or cracking. The walls of poured in place concrete tanks shall be at least 150 mm thick and adequately reinforced.

5.15.2.12. Dosing Devices

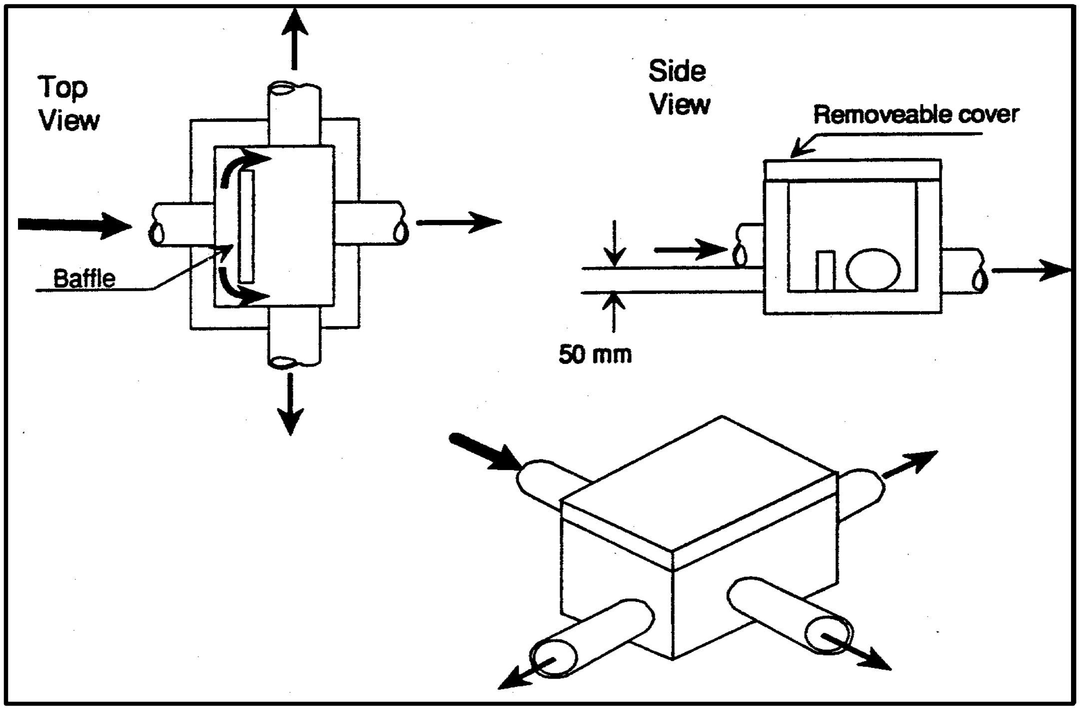
If a distribution field is being used and is has a length greater than 150 m, the tank shall have a pump or siphon designed to discharge a volume of sewage equal to approximately 70% of the volume of the distribution field. A siphon or pumped dosing system may be used depending on the circumstances. The dosing chamber shall have sufficient volume to provide the desired dosing volume, plus a reserve volume if it is a pumped system. In the case of a pumped system, the reserve volume is the volume of the chamber between the high water alarm switch and the invert of the inlet pipe. Also, in the case of a pumped system, an alarm system must be provided to indicate that a problem has occurred.

5.15.3. Distribution Boxes

Adequate frost protection must be provided. The distribution box shall be installed level to ensure that there is an equal splitting of flows. See Figure 5.4 for distribution box details.

Note: there have been some operational problems with distribution boxes in Newfoundland and Labrador.

Figure 5.4
Distribution Box Detail



5.15.4. Tile Field

5.15.4.1. Location

The following are minimum clearance distances:

1. 30 m to the nearest dug well or other source of water supply;
2. 15 m to a drilled well, which has a casing to at least 7.5 m below ground;
3. 15 m to a building;
6. 7.5 m from water service line;
7. 15 m from any watermain;
8. 3 m to a property boundary; and
9. 30 m to any lake, stream or pond.

5.15.4.2. Design

1. For undisturbed soil only, a tile field serving a building or structure other than a private dwelling having four bedrooms or less shall have a total length in metres of distribution pipe which shall be greater than or equal to the value determined from the following formula:

$$L = 0.011Q\sqrt{T}$$

Where: L = total length of distribution pipe (m);
Q = total daily sewage flow (L); and
T = percolation time (min).

2. For imported fill percolation tests see Table 5.31;

3. The minimum percolation time for imported fill shall be 5 minutes;

4. The bottom of a tile field shall be at all points at least 0.9 m above the maximum elevation of the groundwater table or rock or other impervious stratum located in the area of the bed.

5. Absorption trenches shall be at least 450 mm in width and shall be between 600 mm and 900 mm in depth.

6. Distribution pipe shall be located l.8 m or more apart.

7. Distribution pipe shall have a diameter not less than 100 mm and a uniform slope of between 0.33% and 0.5%, except if a siphon or pump is to be used when a uniform slope of between 0.25% and 0.33% shall be used.

8. Distribution pipe shall be surrounded by gravel or broken stone 19 mm to 28 mm in size from a level 150 mm below the bottom of the pipe to a level 75 mm above the pipe.

9. Distribution piping shall have the ends interconnected or capped.

10. Filter fabric shall be placed over the stone or broken gravel to prevent the migration of silt into the absorption trenches.

11. See Figure 5.5.

Table 5.31
Commonly Used Fill Materials and their Design Infiltration Rates

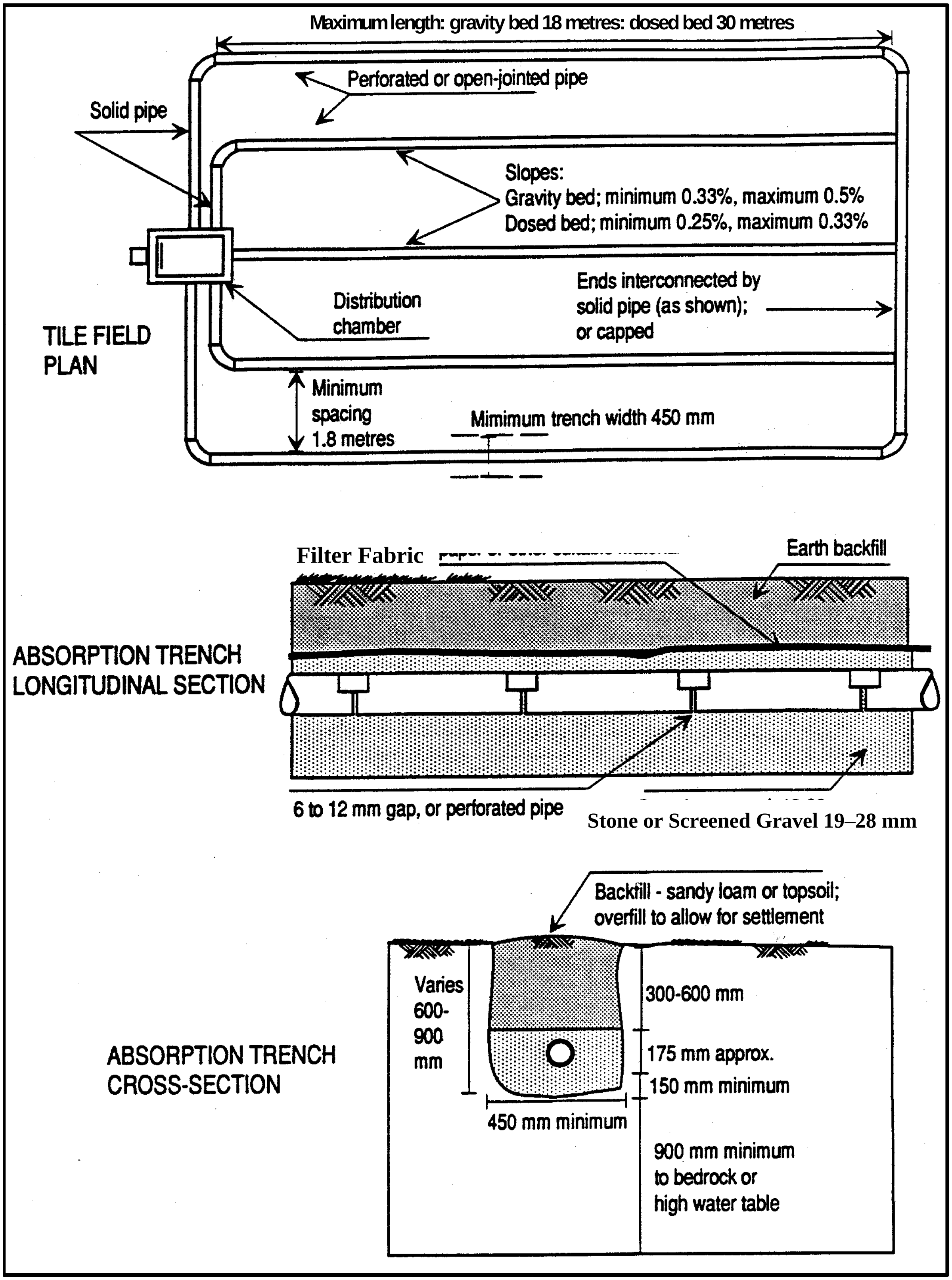
Fill Material	Characteristics[1]	Design Infiltration Rate ($L/d/m^2$)
Medium Sand	> 25% 0.25 – 2.0 mm < 30 - 35% 0.05 – 0.25 mm < 5 - 10% 0.002 – 0.05 mm	49
Sandy Loam	5 – 15% Clay Content	25
Sand/Sandy Loam Mixture	88 – 93 % Sand 7 – 12% Finer Grained Material	49
Bottom Ash	-	49

[1] Percent by weight

5.15.4.3. Construction

Grading of the site should be completed before the system is installed. The interface between the existing ground and the absorption field shall be scarified. After installation, heavy equipment and vehicular traffic shall be excluded from the tile field area.

Figure 5.5
Tile Field Details



5.15.4.4. Site Appraisal and Soil Percolation Tests

The most important step for subsurface disposal is the appraisal of soil and site conditions. Test pits should be dug throughout the site to ensure a complete knowledge of the various factors affecting the design. Observations should be made of soil type and conditions, the distance to groundwater, the distance to bedrock or other impervious stratum and any other factors, which influence the design.

The suitability of the soil for absorbing the liquid shall be estimated by soil percolation tests.

A percolation test shall be conducted as follows:

1. An excavation shall be made in the soil at the site where the leaching bed is to be located;

2. The excavation shall have the following dimensions:
   a) The diameter shall be between 100 mm and 300 mm; and
   b) The depth shall be the distance between the ground level and the bottom of the proposed leaching bed.
3. All loose material and smeared clay shall be removed from the sides and the bottom of the excavation;

4. The bottom of the excavation shall be covered with 50 mm of sand or fine gravel;

5. Clear water shall be poured into the excavation to a depth of at least 300 mm;

6. Additional clear water shall be added as may be necessary to maintain a depth of water of at least 300 mm in the excavation until the soil in the area of the excavation has become swollen and saturated with water and the water being added to the excavation seeps away at a constant rate; and

7. The average time in minutes required for the water to drop 25 mm shall be determined.

8. Refer to Table 5.32.

9. See Figure 5.6.

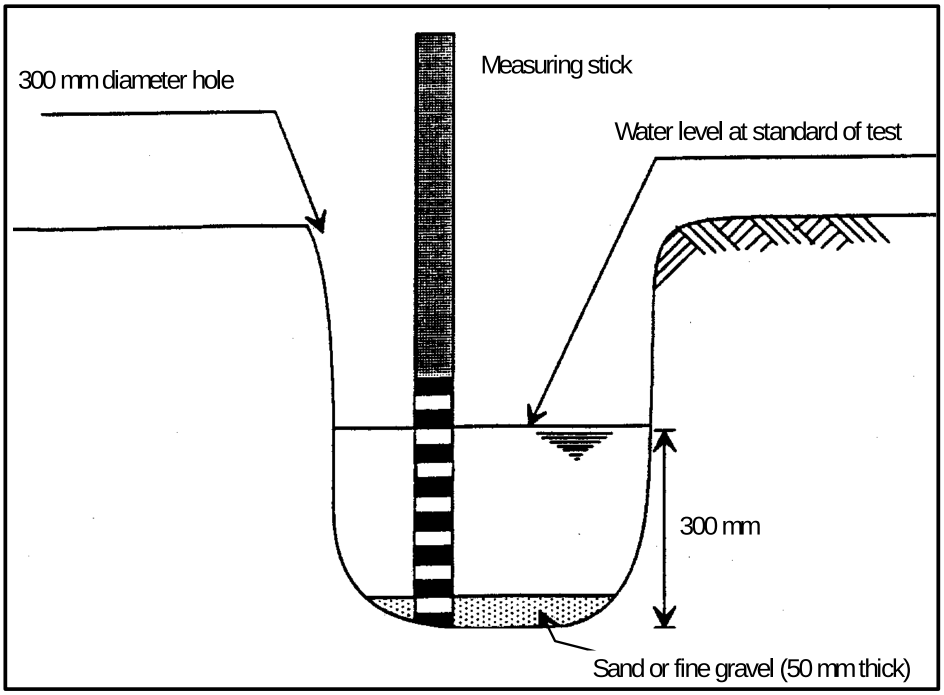
Table 5.32
Required Absorptive Area

Time for 25 mm fall (minutes)	Application Rate (m^3/m^2d)
0 – 5	0.117
6 – 7	0.103
8 – 10	0.083
11 – 15	0.064
16 – 20	0.054
21 – 30	0.039
Over 30	Questionable Suitability

5.15.5. Other Distribution Methods

In soil that is relatively impermeable tile field trenches are not satisfactory. They become more expensive as the soil permeability decreases and may not be feasible when the percolation time exceeds 30 or 40 min. Therefore when absorption trenches are impractical, the possibility of treating septic tank effluent using other disposal methods such as sand filter trenches, mound system, contour trench, peat beds, constructed wetlands, etc. should be considered.

Figure 5.6
Percolation Test Arrangement



5.16. Holding Tanks

5.16.1. Policy

Before a permit is issued for a holding tank, the owner should have an agreement with a licensed contractor for the disposal of the holding tank waste.

5.16.2. Tank Construction and Installation

The following are guidelines for the construction of any tank used in a sewage system for collecting, holding or storing sewage:

1. The tank shall be constructed of concrete, steel, fibreglass, reinforced plastic, or such other material, as may be approved.

2. The tank installed, assembled or constructed shall:
   a) Be carefully made, exhibit craftsmanship and true quality, be sound, durable and thoroughly capable of satisfactory and trouble-free service;
   b) Have such wall thickness, reinforcing and strength as is necessary to meet the requirements of use in service and any stresses to which it may be subjected prior to or during installation; and
   c) Be of watertight construction.

3. Materials used in the construction of the tank shall meet the appropriate industry standards and codes applicable to such materials.

4. The tank shall be constructed or installed on site according to good construction practice or, where specified, the manufacturer's recommendations, and in a manner that will ensure against:
   a) Subsequent settlement of the tank or subsequent uplift due to ground water pressure, that will be disruptive to the sewage system of which the tank is a part;
   b) Damage to the tank or any protective coating during handling and backfilling; and
   c) Damage to the tank due to weather or frost.

5. A prefabricated holding tank shall be constructed so as to meet the requirements for certification by:
   a) The Canadian Standards Association;
   b) The Underwriters Laboratories of Canada; or
   c) An organization accredited by the Standards Council of Canada for certifying products of a type that include such tanks.

6. A tank constructed on-site of concrete shall be poured-in-place and shall:
   a) Comply with the requirements of Canadian Standards Association Standard A23.1;
   b) Have a balanced design of steel reinforcing sufficient to meet the requirements of Clause b;
   c) Have bar or wire reinforcement in accordance with the requirements of Canadian Standards Association Standard A23.3;
   d) Conform to the general requirements appearing in clause 3 of Canadian Standards Association Standard B66 except as otherwise provided herein;
   e) Have top and bottom slabs of a thickness of at least 10 cm in the case of a tank having a capacity of 9000 L and at least 15 cm for a tank having a capacity in excess of 9000 L;
   f) Have sides and ends with a thickness of at least 15 cm in the case of a tank having a capacity of 27 000 L or less and at least 20 cm for a tank having a capacity in excess of 27 000 L; and
   g) have an inlet and outlet of such size as required to accommodate the sewage flows.

The following are guidelines for the construction of a holding tank, which is to be used for the holding or storage of sewage, prior to its collection:

1. The holding tank shall have an apparatus or device installed and kept operating to provide a warning, which is visual or audible or both to indicate when the tank is nearing capacity and should be emptied;

2. The holding tank shall be of a design and construction that will allow it to be sealed in such a manner as to be capable of withstanding internal pressure as specified in Underwriters Laboratories of Canada Standard;

3. The holding tank shall be of a design and construction as will allow the complete removal of solid matter that can be expected to settle in the holding tank;

4. The holding tank shall have an apparatus or device suitable for allowing the contents of the holding tank to drain from or be otherwise removed from the holding tank and any such apparatus or device shall have a suitable lock-out device;

5. The lock-out device shall remain locked or sealed except when it is necessary to unlock or unseal the device for the purpose of repairing, servicing, cleaning or emptying the holding tank and the lock-out device shall not be unlocked or unsealed except by a person engaged in the business of repairing, servicing, cleaning or emptying holding tanks or by a provincial officer;

6. The working capacity of a holding tank shall not be less than 9000 L; and

7. The holding tank shall have an apparatus or device capable of preventing the overflow of sewage from the holding tank.

5.17. Communal Septic Tank Systems

Communal Septic Tank Systems are generally used for the collection and primary treatment of sewage waste from a municipal sewer system. Approval of municipally owned and operated communal septic tank systems, regardless of capacity, is the responsibility of the DOEC, as per Section 36 of *The Water Resources Act*, SNL 2002 cW-4.01.

5.17.1. Location

The following are minimum clearance distances, however local conditions may necessitate greater distance:

1. 15 m from any well, lake, stream or pond;
2. 1.5 m from any building;
3. 3 m from any property boundary;
4. 7.5 m from a water service line; and
5. 15 m from a water distribution main.

5.17.2. Capacity

Tanks will be sized to provide a minimum of 24 hours retention at the design flow capacity to provide solids settling time. Tank liquid capacity will be based on the following criteria:

1. Actual measured flows; **OR**
2. 340 L/capita/day, if using number of persons; **OR**
3. Number of houses x 3.0 x 340 litres, if using number of homes to be connected; **PLUS**
4. Any additional extraneous or other flows, on case-by-case basis.

Examples: 100 persons: 340 L x 100 = 34 000 L (34 m^3)

30 homes: 30 x 3.0 x 340 L = 30 600 L (30 m^3)

In addition to liquid capacity, the tank must provide a minimum of 300 mm storage between the water level and the top of the tank to allow for scum storage.

The minimum liquid depth in the tank shall be 1220 mm.

5.17.3. Solids Retention - Inlets and Outlets

This section specifies requirements for the inlet/outlet device of the communal septic tank and is in addition to section 3.2.3 of CAN/CSA-B66-M90. Inlets and outlets shall be of the centred tee type to permit rodding when necessary. Inlet and outlet pipes shall extend below the liquid level by 360 mm, and extend above the liquid depth by 200 mm.

5.17.4. Maintenance Requirements

While communal septic tanks do not require daily operator attention, regular, periodic maintenance is required to ensure efficient operation. Septic tanks receiving domestic sewage accumulate sludge at a rate of roughly 65 L/capita/year. Maintenance objectives include:

1. Prevention of tank blockages that result in sewer system backups;
2. Prevention of odour problems resulting in complaints;
3. Periodic sludge and scum level measurements to predict removal needs;
4. Prevention of solids discharge to downstream small diameter sewers, disposal beds, and receiving waters; and
5. Removal of sludge and scum when necessary.

5.17.5. Material of Construction

The tank shall be made of sound and durable material. Any fitting, pipe, baffle, device partition, or other component part shall be compatible with the tank. Prefabricated communal septic tanks shall conform to CAN/CSA-B66-M90 or the latest edition in effect at the time of construction. Cast in place concrete communal septic tanks shall conform to CAN3-A23.1-M90 and testing in accordance with CAN3-A23.2-M90 or the latest edition in effect at the time of construction. Cast in place concrete tanks shall have a minimum wall thickness of 150 mm and be properly reinforced.

5.17.6. Top of Tank

The top of the tank shall be at least 300 mm below the finished grade. There should be at least 300 mm clearance between the underside of the top of the tank and the liquid level of the tank.

5.17.7. Bedding

Pre-cast tanks shall be installed according to manufacturer's requirements. Precast/Cast In Place Tanks should be placed on properly designed granular bedding type 2 in accordance with the Department of Municipal and Provincial Affairs Water, Sewer and Roads Master Specification Section 02223.

5.17.8. Access

At least one manhole with a minimum dimension of 600 mm shall be provided in each compartment. The manhole is to extend to the ground surface to facilitate inspection and sludge and scum removal. The manholes shall be located above the inlet and outlet points.

5.17.9. Scum Capacity

Scum capacity for the communal septic tank shall conform to CAN/CSA -B66-M90 Section 3.2.4 or the latest edition in effect at the time of construction.

5.17.10. Effluent Disposal Options

5.17.10.1. Ocean/Marine Discharge

Tank effluent may be discharged to an ocean or marine receiving waters. Outfall location and design will be based on the assimilative capacity of the water. Refer to Section 5.2.15.

5.17.10.2. Exfiltration Beds

Tank effluent may be discharged to an exfiltration bed/leach chamber installed between high tide mark and low tide mark where conditions permit.

5.17.10.3. Communal Disposal Beds

Tank effluent may be discharged to large conventional disposal beds. Reference shall be made to Section 5.15.4.

5.17.10.4. Distribution System

Tank effluent may also be discharged into a small diameter distribution system (SDDS) (see Section 5.2.14.7). The SDDS then transports the effluent to a properly designed area such as a marine outfall, treatment facility etc., or to a conventional municipal sewage collector system. The transport of effluent through the SDDS can normally be achieved through gravity, however on site conditions may necessitate pumping.

5.17.10.5. Other Innovative Means of Disposal

It is recognized that technological advances may present alternative options for disposal of septic tank effluents. The DOEC may review and consider such options. Acceptability will generally be based on site-specific requirements, and will usually require proof of acceptability in other jurisdictions.

5.18. Travel Trailer Dumping Stations

The design of travel trailer dumping stations must take into consideration and make provision for the possibility of sewage being spilled during the dumping process. This should be addressed by installing a sloped concrete pad and adding a potable water source for wash down and hygienic purposes with appropriate provisions to avoid cross contamination with the water source through potential backflow situations.

The use of a sloped concrete pad design will permit spilled sewage and wash down water to be directed to a ground level drain equipped with a hinged cap that is self-closing and vandal proof. When using this design, an approved backflow prevention device must be permanently installed on the potable waterline. The wash down hose should also be permanently attached to a wooden or steel post in order to restrict its movement to the immediate area of the dumping station. To prevent parking lot drainage from entering the dumping station, the concrete pad for the dumping station should be raised above the adjoining parking lot or roadway.

5.19. Disinfection of Sewerage

Disinfection of sewerage treatment plant effluent shall be required in all cases, except where otherwise approved by the DOEC.

The design shall 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 flow rates, pH of waste, disinfectant demand rates, current technology application, cost of equipment and chemicals, power cost, and maintenance requirements.

5.19.1. Forms of Disinfection

Chlorine is the most commonly used chemical for wastewater effluent disinfection. The forms most often used are liquid chlorine and calcium or sodium hypochlorite. Other disinfectants, including chlorine dioxide, ozone, bromine or UV may be accepted by the DOEC in individual cases. If chlorination is utilized, it may be necessary to dechlorinate if the chlorine level in the effluent would impair the natural aquatic habitat of the receiving body of water. The use of chlorine capsules may be considered for small systems.

5.19.2. Chlorination

5.19.2.1. Design Guidelines

5.19.2.1.1. Mixing

The disinfectant shall be mixed as rapidly as possible, with a complete mix being completed in 3 seconds. This may be accomplished by either the use of a turbulent flow regime or a mechanical flash mixer.

5.19.2.1.2. Diffusers

A chlorine solution diffuser shall be placed ahead of the contact tank and near the vicinity of the mixing area.

5.19.2.1.3. Contact Time and Residual

A total chlorine residual of 0.5 mg/L is generally required. The required detention time shall be based upon the more stringent of either 30 minutes at design average daily flow or 15 minutes at the design peak hourly flow. The criteria to be used in the design shall be that which provides the largest volume for the contact tank.

5.19.2.1.4. Coliform Levels

Acceptable effluent coliform levels shall be based upon the results of the receiving water study and the receiving water quality guidelines.

5.19.2.1.5. Contact Tank

In order that the chlorine contact tank can provide the required detention, dead zones within the tank must be avoided and the flow through the tank must approach plug flow as closely as possible. Back mixing within the contact tank must be avoided to prevent short-circuiting and the resulting poor disinfection results. Covered tanks are discouraged.

To approach a plug-flow regime, flow channels with length-to-width ratios of greater than 40:1 are required. Length-to-width ratios of 10:1 produce detention times of approximately 70 % of the theoretical residence times. In rectangular tanks, longitudinal baffling to produce long, narrow flow channels with a serpentine flow pattern and with guide vanes at changes in direction is a preferred method.

Since some sedimentation occurs in chlorine contact tanks, provision should be made for periodic sludge removal from the chlorine contact tank(s). The drain should be valved. If it is necessary to take the contact tank out of operation for cleaning, and if short-term discontinuation of disinfection cannot be tolerated due to other critical uses made of the receiving waters, two contact basins shall be provided. In less critical situations, one contact basin will suffice provided that the bypass facilities are equipped with a chlorine application point for emergency disinfection.

5.19.2.2. Chlorination Facilities Design

Refer to Sections 4.2.4 and 4.2.5

5.19.2.2.1. Feed Equipment

For normal domestic sewage, Table 5.33 may be used as a guide in sizing chlorination facilities.

In order that effective disinfection can be maintained at all times, without the need to overdose excessively at low flow periods, the chlorine feed equipment should be paced by the effluent flow rate.

For additional information regarding feed equipment, including capacity, standby equipment, spare parts, etc., refer to Sections 4.2.4 and 4.2.5.

Table 5.33
Chlorine Dosage Requirements

Type of Treatment	Dosage (mg/L)
Raw Wastewater (Fresh)	6 – 15
Raw Wastewater (Septic)	12 – 25
Primary Effluent	5 – 20
Activated Sludge Plant Effluent	2 – 8
Trickling Filter Plant Effluent	3 – 10
RBC Plant Effluent	3 – 10
Tertiary Filtration Effluent	2 – 6
Nitrified Effluent	2 – 6

5.19.2.2.2. Odour Control

Should odour control be a critical factor, additional capacity of a pre-chlorination system to the extent of about 80% of the raw wastewater chlorine demand shall be required during the warm summer days. It is not desirable to split the functions of the chlorinators, especially for large plants. One group shall be designed for pre-chlorination and another for disinfection. In the case of large plants, each group shall be interchangeable to facilitate a standby feature.

Pre-chlorination must be accompanied ahead of the first open structure in the plant and thereby reduce the escape of hydrogen sulphide gas into the atmosphere.

5.19.2.3. Other

For additional information regarding chlorine supply, methods of dosage control, storage and handling, piping and connections, etc., refer to Section 4.2.

5.19.3. Dechlorination

Dechlorination of effluent shall be considered when the receiving water is:

1. Considered to be highly important for the fishing industry; or

2. Ecologically sensitive to chlorine toxicity and susceptible to the adverse effects of chlorine residuals; or

3. Of public health importance.

The decisions regarding use of dechlorination shall be made on a case-by-case basis.

Dechlorination is especially recommended for situations where low coliform densities, as well as chlorine residuals, are jointly required. The most common dechlorination chemicals are sulphur compounds, particularly sulphur dioxide gas or aqueous solutions of sulphite or bisulphate. Pellet dechlorination systems are also available for small facilities. 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, and safety.

5.19.3.1. 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 forms of the compound that are commonly used and yield sulphite (SO_2) when dissolved in water are presented in Table 5.34.

Table 5.34
Dechlorination Chemicals and Required Amounts

Dechlorination Chemical	Theoretical mg/L Required to Neutralize 1 mg/L Cl_2
Sulphur Dioxide (gas)	0.9
Sodium meta bisulphate (solution)	1.34
Sodium bisulphate (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 dissolved oxygen for every 4.0 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.

5.19.3.2. Containers

Depending on the chemical selected for dechlorination, the storage containers will vary from gas cylinders, liquid in a 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.

5.19.3.3. Feed Equipment, Mixing, and Contact Requirements

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 shall include consideration of the operator safety and overall public safety relative to the wastewater treatment plant's proximity to populated areas and the security of gas cylinder storage. The selection and design of sulphur dioxide feeding equipment shall take into account that the gas reliquifies quite easily. Special precautions must be taken when using ton containers to prevent reliquifaction.

Where necessary to meet the operating ranges, multiple units shall be provided for adequate peak capacity and to provide a sufficient low feed rate on turn down to avoid depletion of the dissolved oxygen concentrations in the receiving waters.

The dechlorination reaction with free or combined chlorine will generally occur within in 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.

A minimum of 30 seconds for mixing and contact time shall be provided at the design peak hourly flow or maximum pumping rate. A suitable sampling point shall be provided downstream of the contact zone. Consideration shall be given to a means of reaeration to assure maintenance of an acceptable dissolved oxygen concentration in the stream following sulphonation.

5.19.3.4. Housing Requirements

5.19.3.4.1. Feed and Storage Rooms

The requirements for housing SO_2 gas equipment should follow the same guidelines as used for chlorine gas.

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 4.2.5. 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 treatment unit.

5.19.3.4.2. Protective and Respiratory Gear

The respiratory protection equipment is the same as used for chlorine (see Section 4.2.4). 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 4.2.4.

5.19.3.5. Sampling and Control

Facilities shall be included for sampling the dechlorinated effluent for residual chlorine. Provisions shall be made to monitor for dissolved oxygen concentration after sulphonation when required by the DOEC.

Provisions shall be made for manual or automatic control of sulphonator feed rates based on chlorine residual measurement or flow.

5.19.3.6. Activated Carbon

Granular activated carbon may also 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.

Dechlorination usually is accomplished in fixed downflow beds using gravity or pressure type filters. Regular backwashing is necessary to preserve dechlorination efficiency.

Suggested design criteria for a wastewater dechlorination activated carbon system, based on potable water application., include a wastewater application rate of 2 L/m^{2}s, an empty bed contact time of 15 to 20 minutes with an influent free residual of 3 to 4 mg/L, and an effective carbon bed life of at least 3 years.

5.19.4. UV Disinfection

The following sections describe factors that affect the performance of UV disinfection systems. Systems should be designed to account for these factors.

5.19.4.1. UV Transmission

UV 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 Transmission or Absorbance and it is a function of all the factors that 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.

5.19.4.2. Wastewater Suspended Solids

Some of the suspended solids in wastewater will absorb or reflect the UV light before it can penetrate the solids to kill any occluded organisms. UV light can penetrate into suspended solids with longer contact times and higher intensities, but there is still a limit to the ability to kill the microorganisms. UV systems must be designed based on maximum effluent SS levels.

5.19.4.3. Design Flowrate and Hydraulics

The number of microorganisms that are inactivated within a UV reactor is a function of the multiplication of the average intensity and residence time, as per the following equation.

$$D = I \times t$$

As the flowrate increases the number or size of the UV lamps must be proportionally increased to maintain the same disinfection requirements. An UV disinfection system must be designed for worst-case conditions. The minimum dosage occurs at the maximum flowrate and end of lamp life.

5.19.4.4. Level Control

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. The UV system must be designed for the maximum flowrate. This is especially important if the wastewater treatment plant receives storm water runoff. The UV system must also be designed to operate at the maximum flowrate. During low flow periods, the wastewater has a greater chance to warm up around the quartz sleeves and produce deposits on the sleeves. There is also the possibility of exposing the quartz sleeves to the air. Because the lamps are warm, any compounds left on the sleeves will bake onto them. Water splashing onto these exposed sleeves will also result in UV absorbing deposits. When the flow returns to normal, some of the water passing through the UV unit may not be properly disinfected. The designer must be very careful with the selection of the flow control device for the above situation. Both flow gates and weirs may be used for level control.

5.19.4.5. Iron Content

Iron can affect the UV disinfection by absorbing UV light. Dissolved iron, iron precipitate on quartz sleeves, and adsorption of iron by suspended solids, bacterial floc and other organic compounds, all decrease UV transmittance. Wastewater with iron levels greater than 0.3 mg/L may require pre-treatment to attain the desired disinfection level.

5.19.4.6. Wastewater Hardness

Calcium and magnesium salts, which are generally present in water as bicarbonates or sulphates, 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.

5.19.4.7. Wastewater Sources

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.

5.19.4.8. UV Lamp Life

Low-pressure mercury lamps are rated for 9000 hours of continuous use. Rated average useful life is defined by the UV disinfection industry as the elapsed operating time under essentially continuous operation for the output to decline to 60% of the output the lamp had at 100 hours. The UV system must be designed so that the minimum required dose or intensity is available at the end of lamp life.

Power costs and lamp replacement costs are the two main factors affecting UV maintenance expenditures. Therefore, UV lamps should only be replaced of no other cause for not meeting the disinfection requirements can be found. Examples of other causes are quartz sleeve fouling, decreased levels of UV transmission, or increased levels of suspended solids in the wastewater.

5.19.4.9. UV System Configuration and Redundancy

Once the number of lamps required to meet the required disinfection permit levels has been determined, a system configuration must be developed. This configuration must meet operational requirements such as plant flow variations and redundancy requirements. Redundancy helps insure that the UV system can continue to operate and meet disinfection permits in spite of a subsystem or component failure. It allows regularly scheduled maintenance such as quartz cleaning to be performed at any time.

5.19.5. Ozone

Ozone Generation

Ozone may be produced from either an air or an oxygen gas source. Generation units shall be automatically controlled to adjust ozone production to meet disinfcetion requirements.

5.19.5.1. Dosage

The ozone demand in the wastewater must be satisfied, as evidenced by the presence of an ozone residual, before significant disinfection takes place. Below this dosage there is reduction of oxygen-consuming material.

Because of the form of ozone and its short life, it is necessary that it be step-fed into the wastewater to provide the contact period needed to accomplish disinfection.

Effectiveness of ozone as a disinfectant is relatively independent of pH and temperature values, although a pH of 6.0 to 7.0 appears to be the most favourable range. A dosage of 5 to 8 mg/L is needed to accomplish disinfection of secondary effluent. The amount and characteristics of suspended solids present in the secondary effluent can be used to determine ozone dosage empirically:

$$Ozone\,Dosage = 1.5 + 0.38TSS$$

5.19.5.2. Design Considerations

5.19.5.2.1. Feed Equipment

Ozone dissolution is accomplished through the use of conventional gas diffusion equipment, with appropriate consideration of materials. If ozone is being produced from air, gas preparation equipment (driers, filters, compressors) is required. If ozone is being produced from oxygen, this equipment may not be needed, as a clean dry pressurized gas supply will be available.

Where ozone capacities of 500 kg/day or less are required, air feed is preferred. Modification of the single-pass air feed system should be considered in determining the most economic system for application in wastewater treatment.

5.19.5.2.2. Air Cleaning

Removal of foreign matter such as dirt and dust is essential for optimum performance and life of an ozone device. For small units, cartridge-type impingement filters may be economical. For larger operations, electrical precipitator or combination filters are preferred.

5.19.5.2.3. Compression

Positive displacement rotary-type compressors are preferred for large installations. Internally lubricated units should not be used since oil vapour will permanently impair the water-adsorptive capacity of the driers. Need for standby capacity and flexibility of operation requires the installation of several blower units

The required compressor rating will depend on the pressure drop through the entire system. Generally, a 70 kN/m^2 pressure is necessary to force the air through the coolers, driers, ozonation devices, and the 4.5 to 6 m head of water in the mixing and contact system.

5.19.5.2.4. Cooling and Drying

Pretreatment for reducing moisture in the feed gas stream shall be required.

5.19.5.2.5. Injection, Mixing and Contact

Intimate mixing of an ozone-enriched air stream with the wastewater as well as maintaining contact for an adequate period of time is essential. The major problems to be considered are satisfying the ozone demand, the rapid rise of the gas to the liquid surface of the contact chamber and escape of ozonated air bubbles, and the relatively short half-life of ozone. Consequently, where ozone contact beyond a few minutes is needed, the ozonated feed stream is staged with the amount of ozone for each stage set at a level that can be consumed usefully.

5.19.5.2.6. Controls

The design engineer should be cognizant of the fact that ozone is a toxic gas, and that if compressed oxygen is used as the feed gas, special provisions must be met in its handling and storage. The ozonation process involves a series of mechanical and electrical units that require appropriate maintenance and repair and are susceptible to the same malfunctions as are all such pieces of equipment. Standby capacity normally is provided in all essential components. Information can be obtained from the equipment manufacturer on the metering and alarm systems needed for continuous process monitoring and warning of failure in any element of the process.

5.19.5.2.7. Piping and Connections

For ozonation systems, the selection of material should be made with due consideration for ozone's corrosive nature. Copper or aluminum alloys should be avoided. Only material at least as corrosion-resistant to ozone as Grade 304L stainless steel should be specified for piping containing ozone in non-submerged applications. Unplasticized PVC, Type 1, may be used in submerged piping, provided the gas temperature is below 60°C and the gas pressure is low.

Section 6 – Instrumentation and Controls

6. Instrumentation and Controls

6.1. Water Works

Controls and instrumentation should be appropriate for the plant size, complexity and number of staff and their skills for each plant. To achieve this, the designer should develop a control philosophy that will enable the plant staff to effectively monitor and control the plant and major equipment, the treatment process, water production and; plant wastes.

6.1.1. Measurement List

For plants of 1 ML/d capacity and greater, the following instruments should be provided as a minimum for the relevant processes listed. For smaller plants, pH measurement and fluoride residual may be made by bench testing but all other instruments are appropriate for the relevant processes listed.

1. **Raw Water Instrumentation:**
    a) Low-level switches to shut down the raw water pumps. These should be hard-wired to the starters;
    b) Running and trip indication f or raw water pumps; and
    c) Raw water turbidity, pH, pressure, flowrate, and flow volume.

2. **Rapid Mixer:**
    a) Running and trip indication.

3. **Flocculators**
    a) Running and trip indication; and
    b) Speed if variable speed type.

4. **Solids Contact Clarifiers**
    a) Recirculator speed indication;
    b) Running and trip indication;
    c) Level indication;
    d) Blow down valve status; and
    e) Turbidity and pH following clarification.

5. **Softening**
    a) If lime softening is used, pH following recarbonation; and
    b) Recarbonation CO2 feed status.

6. **Filter Instrumentation**
    a) Turbidity on each individual filter effluent and filter to waste. This can be a single instrument for each filter if piping arrangement permits;
    b) For constant rate filters: differential head loss across the filter media;
    c) Filter flowrate;

d) Where the backwash sequence is automated, provide open and close limit switches or position on all filter valves and status on backwash equipment; and
e) Filter run-time.

7. Backwash Instrumentation
a) Running and tri p indication f or backwash pump(s);
b) Running and tri p indication for air blowers (if air scour is used); and
c) Backwash flowrate and flow total.

8. Clearwell and Distribution Pump Instrumentation
a) Level indication for clearwell and other tanks;
b) Low-level switches to shut down the distribution pumps. These should be hard wired to the motor starters;
c) Turbidity, chlorine residual, fluoride residual (if fluoridation is practiced), pH, pressure, flowrate, and flow total on plant discharge; and
d) For variable speed pumps, indicate the pump speed.

9. Chemical Systems
a) Running and tri p indication for chemical loading, batching and pumping equipment;
b) Low and high level indication in storage bins, silos or tanks;
c) Level indication for tanks;
d) Weigh scales for hydrofluosilicic acid day tanks or storage if no day tank is used;
e) Weigh scales for gaseous feed chemicals such as chlorine or sulphur dioxide;
f) Speed indication on variable speed pumps;
g) Rotameters for carrier water feed systems; and
h) Chemical feed flowrate is desirable but not mandatory.

10. Miscellaneous Instrumentation
a) Run time meters on all pumps and major electrically driven equipment;
b) Speed, run time, oil pressure and temperature gauges, fault signal switches and manual start and shut down on engines;
c) Where the plant is automated or operated remotely from either within the plant or outside, provide open and close limit switches or position on all major valves, status on all major equipment and security instruments including door switches, building temperature switches and smoke alarms; and
d) Any additional instrumentation recommended by equipment manufacturers.

6.1.2. Degree of Automation in Plant Control

The control system may be manual or automatic or a combination thereof. Regardless, the system should be designed to promote energy efficiency, conserve water, and reduce waste while meeting the treated water quality standards and demands under all anticipated conditions. Discuss the operating philosophy with plant staff and owners to determine the appropriate degree of automation.

In the case of a manual system, all equipment is started and stopped by the operator, all backwash sequences and other process operations are controlled by the operator, and chemical and pump rates are manually adjusted. This requires that the plant be manned continuously while in operation, perhaps with more than one operator.

In the case of an automatic system, all equipment is started and stopped by the control system, with chemical feed rates and pump rates adjusted automatically to maintain the system levels, discharge pressures, etc. This may allow unattended plant operation or operation with a single operator, but requires a more complex and expensive control system, with associated maintenance. Provide the ability to manually operate all equipment.

For systems with rapidly varying raw water conditions, fully automated plants should not normally be considered.

6.1.3. Alarms and Status Indication

All alarms must be latched until the Operator has acknowledged them. If the alarm is indicated by a lamp, it must flash until acknowledged then remain steady until the alarm clears. If it is indicated on a computer screen, an appropriate colour code or symbol must be used to indicate for each alarm whether it has been acknowledged. Automated systems should log the time at which the alarm occurred, the time it was acknowledged and the time it cleared. Logs may be printed on paper or recorded electronically.

Valve and equipment status should use a consistent method of symbols and colours, whether the status is indicated through lamps or on a colour computer screen. The colour-coding scheme should be consistent with any existing equipment displays elsewhere in the plant.

As a minimum, the following alarms should be provided:

1. High turbidity on the raw water, clarifier effluent (if applicable), filter effluent, and plant discharge;
2. High and low pressure on the raw water line;
3. High flowrate on the raw water line;
4. High and low level in clarifiers or flocculators;
5. High torque on solids contact clarifier recirculator and rake;
6. High torque on flocculators;
7. High level in filters;
8. High and low level in chemical storage tanks;
9. High and low chemical feed rates;
10. High flowrate on each filter individually (also low flowrate on declining rate filters);
11. High and low levels in each clearwell, pumpwell, and reservoir;
12. High and low pH on the raw and treated water (if on-line measurements are provided);
13. High and low chlorine residual on the plant discharge (where on-line measurements are provided);
14. High head loss on the filters (if constant rate type);
15. Trip or failure to run on each pump;
16. High and low pressure on the plant discharge line;

17. High flowrate on the plant discharge line;
18. Chlorine gas detection in the chlorine storage, metering and injector rooms;
19. Chlorine scale low weight (where scales are equipped with transmitters); and
20. Valve operation failure (where valves are provided with limit switches).

More alarms may be required where additional treatment processes are provided. Alarms should be provided for all control system interlocks that can shut down equipment or systems. n plants that are left unattended for periods of time, an automatic alarm dialler should be provided.

6.1.4. Control Equipment (Automatic Systems)

Automatic systems should use either Programmable Logic Controllers (PLC's) or a Distributed Control System (DCS). The operator interface may be in the form of traditional control panels (ie: lights, gauges and switches), electronic control panels (with text and/or graphics) and computers.

Digital communication between components of the control system must be reliable and self-monitoring. The communication protocol must meet the following requirements:

1. It must include error checking and reporting, to ensure that data is correctly transferred from one component to another;
2. The components of the system must detect the failure of the communication system (either between individual components of the system or between the system and the operator); and
3. It must be compatible with a variety of manufacturer's instruments and equipment, in order to allow for expansion of the system.

If a DCS is provided, the communication protocol will be proprietary and the manufacturer should be consulted regarding reliability, error checking, and the possibility of connecting other manufacturer's equipment to the network.

If PLC's are used, the communications protocol should use one of the widely accepted industry protocols such as, but not limited to: Modicon MODBUS, Allen-Bradley DATA HIGHWAY and TCP/IP Ethernet.

The operator interface may consist of a local hard wired control panel or mimic, character based input/output panel, personal computer or workstation depending on system size, process complexity, control system functions and operator interface manufacturer. Where personal computers or workstations are used, select the hardware based on reliability, software compatibility, vendor support and suitability for continuous operation in the plant environment. The operator interface software may provide the operator with interactive control and monitoring of the plant, handle and annunciate alarms, log and trend events and process variables and generate the required reports. Process control and logic should be performed by the PLC or DCS and not the operator interface computer or workstation.

6.1.5. Field Instruments

6.1.5.1. Level Instruments

Where access to the top of the reservoir is convenient (such as in a clearwell), ultrasonic level transmitter should be used. Where access to the bottom of the reservoir is convenient (such as at a tower or above-ground reservoir), a pressure transmitter should be used as a level-sensing device.

6.1.5.1.1. Ultrasonic Level Measurement

The ultrasonic level transmitter fires a “sonar” signal toward a surface, such as the surface of the water in a well, and measures the time required to receive an “echo” in order to determine the level of the liquid.

The ultrasonic transducer should be installed so that it is protected from damage, there are no obstructions between the transducer and the water surface, and it is accessible for calibration and maintenance.

1. The transducer should be installed in the top of a stilling well to prevent turbulence from producing errors in the reading. The stilling well should be a continuous length of pipe, either PVC or steel of sufficient diameter and without couplings or fittings that could reflect a sonic echo back to the transducer thus giving a false reading.
2. The well must extend from a convenient height above the high-water line, at which level the transducer will be installed, to the low-water line.
3. Consider the transducer’s “blanking distance”, inherent its design, and ensure that the transducer is mounted high enough above the high-water line so that it will properly read the highest water level anticipated.
4. Several holes should be provided in the side of the stilling well near the bottom for water to enter the well. The holes must be large enough to prevent clogging if silt is present.
5. The controller and display should be located where it can be conveniently read by the operator.
6. Where the air temperature between the transducer and the liquid surf ace is not constant (this is usually the case), provide a temperature measurement for the controller in order that it can compensate for the speed of sound travel through the air, and correct for temperature variations. Note that some manufacturers include the temperature sensor in the transducer itself, while some provide a separate temperature probe.

6.1.5.1.2. Pressure-Sensing Level Transmitter

The pressure-sensing level transmitter reads the head of a column of liquid and transmits a signal proportional to the level of liquid.

1. The level transmitter should be installed as near as practical to the bottom of the tank being measured, so as not to introduce a zero offset in the reading.
2. A block and bleed valve should be provided on the pressure line so that the transmitter can be calibrated for zero level, and can be removed from service.
3. If the pressure-sensing line is small in diameter (12 mm or less), clamp it to supports or walls to provide adequate support.

4. If the transmitter is equipped with an integral display, the transmitter should be located so the display is clearly visible. If no display is provided, and the head being measured is high enough (100 kPa or higher), consider installing a pressure gauge in addition to the transmitter as a backup and calibration aid.

6.1.5.2. Flow Instruments

On line, flow meters should generally be one of the following types:

1. Turbine (or nutating disk);
2. Magnetic; and
3. Ultrasonic (either transit-time or Doppler).

All of these types of instruments can be equipped to provide both flow rate and flow total measurements.

Price, line size, flowrate, flow range, required accuracy and water quality will dictate the election of the type of instrument. The following are some general guidelines:

1. Where considerable silt is present (as in many raw waters), either a magnetic or a Doppler-type ultrasonic meter should be used. Turbine meters will wear rapidly and are not practical. Transit-time meters may not operate properly if there is considerable silt (consult the manufacturer).
2. An ultrasonic meter is generally more economical than a magnetic type on lines of 300 mm diameter and larger.
3. For, on lines of very low flowrate (less than 0.3 m/sec), turbine or magnetic flow meter is recommended. Where chemicals are present in the water, check with the manufacturer to ensure that the meter will not suffer damage.
4. Corrosion and abrasion resistant linings should be considered for these applications. Regardless of the meter type, the minimum flow velocity should be within the specified range of the meter.
5. For, on lines of high flow velocity (higher than 5 m/sec), magnetic or ultrasonic flow meter is recommended. Regardless of the meter type, the maximum flow velocity should be within the specified range of the meter.
6. Where the water is free of solids and bubbles (as is the case on a potable distribution line), Doppler-type ultrasonics will not operate; a transit-time type should be used.

6.1.5.2.1. Turbine Flow Meters

Turbine flow meters determine the flowrate by reading the rotating speed of the turbine, which is immersed in the fluid. A flow totalizer is almost always included, and a flow transmitter is usually available. Because they totalize volume without power, they will continue to operate during power failures and because they will operate without any configuration on the part of the user or operator they are often used where ease of use and maintenance are essential.

1. Where debris may be present in the water, such as in a raw water intake, a screen filter (such as a Y-type strainer) upstream of the meter should be provided.

2. A continuous straight run of piping upstream of the meter, 10 pipe diameters if possible should be provided, to produce a smooth flow profile through the meter. This will minimize errors in the reading.

3. If it is not possible to provide at least five diameters of straight piping upstream of the meter, install straightening vanes in the pipe immediately upstream of the filter. Even with vanes, the accuracy of the meter may be compromised.

4. Five diameters of straight piping should be provided downstream, if possible.

5. If a totalizer or flowrate display is provided, they should be located so that the display is easily read.

6.1.5.2.2. Magnetic Flow Meters

Magnetic flow meters operate by applying a magnetic field around the flowing liquid and reading the voltage produced on a pair of immersed electrodes.

1. The manufacturer should be consulted regarding electrode material and liner material. The meter should operate with silt, chemicals, etc.

2. It is essential that the pipe be full of water at all times; the meter will not operate with large air bubbles in the pipe. Some small bubbles, such as are found downstream of pumps, can be tolerated.

3. A continuous straight run of piping should be provided upstream of the meter, 10 pipe diameters if possible, to produce a smooth flow profile through the meter. This will minimize errors in the reading.

Because magnetic meters read the total voltage produced across the full width of the pipe, some averaging is provided, and this makes them more resistant to turbulence than either turbine or ultrasonic meters. Less than 10 pipe diameters straight run upstream may compromise the accuracy of the meter.

1. Five diameters of straight piping should be provided downstream if possible.
2. Magnetic meters should be supplied with flowrate and flow total displays; the controller should be installed so that the display is easily read.

6.1.5.2.3. Ultrasonic Flow Meters

Ultrasonic flow meters operate by firing a sonic “pulse” through the pipe wall into the flowing liquid. A transit-time meter uses two transducers, one mounted upstream of the second, and measures the difference in travel time for a pulse from one transducer to the other. A Doppler type measures the difference in the frequency received by the transducer as the sonic pulse reflects off particles or bubbles in the liquid. In either case, the difference is directly proportional to the velocity of the liquid.

1. The manufacturer should confirm that the flow meter will operate with the pipe wall material and thickness expected.

2. Ultrasonic flow meters should not be installed where the pipe will contain large bubbles or air pockets; the sonic pulse will be disrupted so that the meter won't operate.

3. Ultrasonic meters are sensitive to the flow profile; at least five pipe diameters of straight piping should be provided (ten pipe diameters recommended) between the meter and an upstream elbow or other hydraulic disturbance.

6.1.5.3. Water Quality Instruments

The most frequently used water quality measurements are turbidity, pH, and chlorine residual. On-line turbidity measurement is relatively inexpensive and should be provided in any plant, on the raw water, flocculator or clarifier effluent (if applicable), each filter effluent, and final plant discharge lines. In larger plants, on-line pH and chlorine residual are generally used, but these can be done though lab tests in smaller plants.

6.1.5.3.1. Turbidity Instruments

Turbidity instruments usually measure the degree to which a beam of light is transmitted or scattered as it passes through a sample of the liquid being measured. A small constant flow of liquid is required to pass through the turbidimeter. It is important that the liquid be free of bubbles, which would scatter the light and produce an erroneously high reading.

1. A transmissive type flow meter should be used for low-turbidity applications such as treated water (turbidity range 0-100 NTU) and a surface-scatter model for high-turbidity applications such as raw water (range 0-5000 NTU).

2. A needle valve and rotameter should be provided to adjust the flowrate through the turbidity meter so that it falls in the range required by the manufacturer. If necessary, a pressure reducing valve should be installed upstream of the needle valve to make the flowrate adjustment easier.

3. The liquid stream is not affected by the turbidimeter; it may be returned to the process or discharged to waste.

4. The sensor element should be located as near to the sample point as practicable to minimize lag time. Where the water contains settleable material, the sample line velocity should be high enough to prevent sedimentation in the line. The use of clear piping should be avoided to reduce the possibility of algae growth.

5. Where a sample line may become plugged by silt, as in a raw water measurement, a manual flush valve should be provided with pressurized plant water to flush the silt either backward into the process line or to waste, as required. A block valve should be provided for the turbidimeter to protect it from the high-pressure f lush water.

6. The sensor element may be mounted some distance from the controller. The controller should include a display and should be installed so that the display is easily read.

6.1.5.3.2. pH Instruments

pH is read by the measurement of an electric potential generated at a pair of electrodes, which are wetted by the sample stream. All pH instruments use a buffer solution, which is generally pumped to the electrodes in very small volumes by the controller. The solution must be replenished at intervals.

1. A needle valve and rotameter should be provided to adjust the flowrate past the electrodes so that it falls in the range required by the manufacturer. If necessary, a pressure reducing valve should be installed upstream of the needle valve to make the flowrate adjustment easier.

2. If the sensor element could become clogged with silt, a filter should be provided upstream. The sensor elements are generally very fragile, so flush lines should only be provided where the electrodes can be completely removed from service during flushing.

3. The liquid stream is contaminated with buffer solution during the measurement; the stream should be discharged to waste.

4. If the controller includes an alarm contact to warn of low buffer solution level, the contact should be tied into the alarm system to remind the operator to refill the controller.

5. The sense element must not be mounted far from the controller because of the very low-level signals involved. If necessary, the sample line should be routed to a location where the sense probe and the controller may be located near each other.

6. The controller should include a display, and should be installed so that the display is easily read.

6.1.5.3.3. Chlorine Residual Instruments

Chlorine residual measurements fall into two categories: amperometric, which measures a potential generated at three electrodes, and polarographic, which measures a colour change when an indicator is added to the liquid sample. Both types of instruments require periodic refilling with buffer or indicator solution.

1. A needle valve and rotameter should be provided to adjust the flowrate past the electrodes so that it falls in the range required by the manufacturer. If necessary, a pressure reducing valve should be installed upstream of the needle valve to make the flowrate adjustment easier.

2. The liquid stream is contaminated with buffer solution during the measurement; the stream should be discharged to waste.

3. If the controller includes an alarm contact to warn of low buffer solution level, the contact should be tied into the alarm system to remind the operator to refill the controller.

4. The sense element must not be mounted far from the controller because of the very low-level signals involved. If necessary, the sample line should be routed to a location where the sense probe and the controller may be located near each other.

5. The controller should include a display, and should be installed so that the display is easily read.

6. Because chlorine measurements are usually limited to treated waterlines, it is not necessary to install flush lines or filters to protect the instrument from debris.

6.1.5.4. Pressure Instruments

Pressure may be simply indicated on a gauge or transmitted (and optionally indicated as well) by a transmitter.

6.1.5.4.1. Pressure Gauges

Pressure gauges are available to read both differential and single-ended pressure. By far the most common measurements are single-ended, although differential gauges are used to read head loss on water and air filters.

1. Where a pressure gauge is reading a pressure produced by a pump (normally required) the gauge should be protected from vibration by filling it with either silicone liquid or glycerine. Silicone should be used if the ambient temperature will fall below -30°C.

2. The range of the gauge should be chosen so that it will normally operate at one-half to two-thirds of scale at normal design pressure; the gauge should not be operated full-time near the top end of the scale. This will provide some safety margin on over-pressure as well as prolonging the life of the gauge.

3. The gauge should be installed where the lens will not get damaged and where it can be read easily. Choose a gauge with a top-mounted stem where it will be installed near the ceiling so that the dial will read right-side up.

4. For water applications (both raw water and treated) a bronze or 316 stainless steel bourdon tube mechanism should be used. For applications on chemical lines, the manufacturer should be consulted for compatibility between the process and the gauge material.

5. On corrosive liquids and processes containing solids, or where the gauge material is not compatible with the process, an isolating diaphragm should be used between the process sense line and the gauge to protect the gauge.

6. A block and bleed valve should be installed between the process and the gauge, or between the process and the diaphragm, to take the gauge out of service.

6.1.5.4.2. Pressure Transmitters

Pressure transmitters are available to read either differential or single-ended pressures. The single-ended type may read either gauge pressure (the pressure relative to the atmosphere) or absolute pressure (relative to a vacuum). Absolute pressure measurements are not common. Differential measurements are commonly used to determine when a filter needs washing.

1. The range of the transmitter should be chosen so that it will normally operate at one-half to two-thirds of scale at normal design pressure; the transmitter should not be operated full-time near the top end of the scale. This will provide some safety margin on over-pressure as well as prolonging the life of the sense element in the transmitter.

2. For water applications (both raw water and treated) a bronze or 316 stainless steel sensing diaphragm should be used. For applications on chemical lines, the manufacturer should be consulted for compatibility between the process and the diaphragm material.

3. On corrosive liquids and processes containing solids, or where the sense diaphragm material is not compatible with the process, an isolating diaphragm should be used between the process sense line and the transmitter to protect the transmitter.

4. A block and bleed valve should be used between the process and the transmitter, or between the process and the isolating diaphragm, to take the transmitter out of service, and to facilitate calibration.

5. Where a pressure gauge is not installed on the same line as the transmitter, an integral display should be provided on the transmitter for local indication.

6.1.6. Process Controls

6.1.6.1. Pumping Systems

Regardless of the function of the pumping system, its control will normally be achieved through monitoring level, flow and/or pressure. The choice of control parameter(s) will depend on the system's function and features.

Controls and monitoring for the following systems is discussed:
1. Raw Water Pumping; and
2. Finished Water Pumping.

6.1.6.1.1. Raw Water Pumping

Raw water pumping is normally controlled by flow, since this sets the production rate of the treatment processes. Typically, this is achieved manually, by selecting the number of raw water pumps operating. If there are variable speed pumps, these will be controlled by flow, with their speed, and output, controlled to match a manually selected flow set point.

Alternatively, pumps may be controlled by level in the plant's treated water clearwell storage reservoirs, or in one of the open unit processes. At selected level set points, a falling water level will bring on another pump, and rising water level will shut down a pump. If variable speed pumps are used, an analog level signal can be used to control pump speed, and hence it's output, and therefore maintain water level within a selected operating band.

Raw water pumping rate should be varied gradually, if possible, and only when necessary. Flowrates through the treatment processes should preferably be kept steady, as this will achieve better and more consistent treatment and water quality. This may be achieved by setting the raw water pumping rate, and hence the plant production rate, to meet the anticipated demand for the day.

On the suction side of the pump(s), pressure is monitored by level or pressure indicator. The pressure/level-measuring device will initiate low level (or pressure) alarm, and low-low level (or pressure) alarm and shut down of pumps, to protect the pumps from cavitation damage or running dry. Where the raw water source exhibits or is subject to rapid increase in free surface elevation (F.S.E.), the pressure/level-measuring device should initiate high level alarm and raw water pumps and possibly the whole plant should shut down for flood protection.

Flow monitoring may be provided on either the suction or discharge side of pumps; normally on the discharge side. Flow monitoring should indicate flowrate, and accumulated flow volume. Coagulant and pre-disinfectant (if used), should be flow paced to the flowrate signal. Pressure on the pump discharge should be monitored. The combination of flow and pressure will serve to monitor pump performance. If the pump discharge flowrate is to be controlled by a modulating valve, pressure should be monitored upstream of the valve to ensure pumps are operating within the normal process operating range, and also to ensure they are not operated outside their allowable envelope. High pressure and low pressure set points should be provided to initiate an alarm condition; high-high and low-low pressure set points will initiate pump shut down. Likewise, monitoring the valve position helps to ensure that the valve can be operated within its working range.

Flow splitting is required where two or more process trains are used. This may be through separate flow meters and flow control valves or through flow splitter boxes employing weirs.

6.1.6.1.2. Finished Water Pumping

Finished water pumping control should ensure that varying demand from the distribution system can be met while maintaining adequate pressure in the distribution system. This will be achieved by controlling flow or pressure, depending on the distribution system into which it feeds.

Flow control may be used in larger systems when the control system is essentially manual, and the distribution system has sufficient storage to accommodate the difference between varying demand and the selected pumping rate. This essentially comprises manual selection of the number of operating pumps, and will require continual operator monitoring and supervision or an automatic system to ensure distribution storage is not depleted or overflowed.

Monitoring discharge pressure is a common approach in controlling small and medium systems. Pump discharge pressure set points will start or stop pumps in a pre-selected sequence. Increased demand in the distribution system will result in falling pressure, when pressure reaches the low pressure set point, it will initiate starting the next duty pump, thereby restoring pressure to within an acceptable range. If pressure continues falling and again reaches the low pressure set point, the next duty pump will start, and so on.

On rising pressure, a pump will be shut down when the high pressure set point is reached. If pressure continues to rise again, the next pump in the sequence will drop out. The pressure set points must be selected to ensure distribution system pressures remain within acceptable limits; the pumps must be selected to ensure they can operate over the range of the operating set points.

Variable speed pumps may be used in finished water pump systems. Multiple pumps are still needed, but fewer units can be used to cover the same flow range, if some or all are variable speed. The most important advantage of variable speed pumping is the ability to maintain a constant discharge pressure into the distribution system. Control of variable speed pumps will be by pressure. Pump speed and output will vary in response to a drift in discharge pressure from the selected set point. A drop in pressure below set point will initiate incremental pump speed increases until pressure set point is restored. A rise in pressure above set point will initiate incremental pump speed decreases until pressure set point is restored. When maximum (minimum) pump speed limit is reached and pressure set point has still not been restored, another pump will start (stop), and the variable speed unit will ramp down (up) until pressure set point is restored. Correct selection and sizing of the pumps is vital to ensure the speed of the pumps remains in the recommended range.

Regardless of the control system, pressure (or level) should be monitored in the suction side to provide alarm and shut down or low pressure (level) and low-low pressure (level).

Discharge flowrate should be monitored continuously, and the accumulated volume recorded. Flowrate will be used to control the feed rate for secondary disinfectant, and where applicable, corrosion control chemicals, and pH control chemicals. Discharge pressure monitoring will also provide alarm on low or high pressure, and pump shut down on low-low or high-high pressure.

6.1.6.2. Treatment Processes

6.1.6.2.1. Travelling Screens

Two methods may be used to control the operation of travelling screens:

1. Simple manual start/stop, which requires the presence of the operator at the screen in order to start and stop the screen. This method is not recommended where sudden changes in raw water quality could result in heavy debris accumulation on the screens.

2. Automatic activation by differential level or time. This method 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 may be programmed to stop when the differential level is returned to the clean screen value, the final stop should be controlled using a sensor

to determine cycle completion (i.e. limit switch, proximity sensor, timer). In addition, a timer should be provided to initiate a cleaning cycle at regular intervals regardless of actual head loss. When this method is employed, there should be an alarm signal with a head loss set at a point higher than the automatic start of the travelling screen.

6.1.6.2.2. Chemical Feed Systems

6.1.6.2.2.1. Liquid Chemical Feed

The chemical dose rate should be flow paced to the plant flow in the part of the process that the chemical is to be injected into. Two methods are typically used to achieve this: metering pump, or flowmeter and flow control valve on the chemical feed:

1. Metering Pump Feed Control - Positive displacement type (diaphragm, peristaltic or progressive cavity) pump should be used. The output of the pump is directly controlled by a 4-20 mA signal from the flow transmitter on the plant flowmeter. On plant shutdown, the flowmeter (usually the raw water flowmeter) will signal the metering pump to stop and a solenoid on the dilution water to close. A load cell or pressure (level) transmitter on the chemical storage tank should provide warning signals when chemical supply is low, and should have alarm and initiate plant shutdown on low-low level.

2. Flow Meter Control - Where the need and justification for a more accurate and positive control system exists, flow meter control may be provided, where the chemical feed rate is controlled by an in-line flow control valve. A PLC receives a 4-20 mA signal from the flow transmitter on the plant flowmeter. Using this dose rate set point, the controller will look at the flowrate on the chemical feed flow transmitter and signals the in-line flow control valve to a position that will control the feed rate to the established set point. On plant shutdown, the controller will signal the in-line control valve to close. Depending on the range of feed rates, multiple flow meters and control valves may be required. As with the metering pump system, low and low-low level alarms/shut-down should be provided on the chemical storage tank(s).

6.1.6.2.2.2. Dry Chemical Feed

Dry chemical feed systems typically include a packaged bulk storage combination feeder and mixer. The feeder can be gravimetric or volumetric, and will be controlled by a 4-20 mA signal from the flow transmitter on the plant flowmeter.

The chemical feeder discharges to a dissolving tank where it is mixed with plant service water to form a solution (slurry) suitable for dosing. Plant service water flowrate is manually set and monitored by flow indicator. The rate needs to be suitable for the range of anticipated chemical feeds and its solubility; the rate may need to be manually adjusted seasonally. For a specific plant service water flowrate, the variation in chemical feed rate creates a corresponding variation in solution strength fed to the process water. The solution is fed via a hydraulic injector (if into a pipeline under pressure), or directly by gravity into open channels or tanks. On plant shutdown a signal from the raw water flowmeter will call for the dry feeder to stop and a solenoid valve on the plant service water feed to the dissolving tank to close. If an injector is used, a solenoid valve on the plant service water will also close.

6.1.6.2.3. Rapid Mixing

Rapid mixing of coagulant and other chemicals is achieved by mechanical or hydraulic mixing. Mechanical mixing may be in a tank or in a pipe and will comprise of a propeller or impeller assembly, usually electric motor driven. Control of the rapid mixer will be simply on or off. The unit should operate continuously whenever the plant is producing.

The use of variable speed or two speed drives can be used to vary mixing energy. If used, the speed may be controlled manually, based on operating experience and/or varied proportion to the flowrate in order to reduce backmixing, if this is of concern.

Hydraulic rapid mix can be in-pipe (static mixer or pumped jet) or free-fall (weir or cascade). These systems require no controls other than on/off for the pumped jet. With the static mixer, it is useful to indicate headloss across the mixer to confirm energy level and also monitor for headloss build-up, which could be indicative of chemical accumulation.

6.1.6.2.4. Flocculation

Flocculation is achieved mechanically or hydraulically in tanks. Mechanical flocculation requires paddles, picket fences or turbines to gently mix the coagulated water to produce a settleable floc for subsequent settlement, (or if the liquid - solid separation process is flotation, to produce a fine pin-point floc for floating). Some processes, such as solids contact clarifiers, integrate flocculation into the unit process. Flocculation requirements should be addressed in terms of the unit process parameters.

Where flocculation is a discrete unit process, it is normally achieved in multiple tanks operating in trains of usually two or three stages. The mixing energy level should be optimized for the water being treated; and will vary as the water quality changes. Jar testing or streaming current monitor will indicate the appropriate chemical dose. Jar testing will also indicate the mixing energy required for optimum flocculation.

For mechanical flocculation, variable speed drives should be used on the equipment. This will allow mixing energy to be adjusted to optimize the process. Speed control will be manual, based on jar tests and operational experience.

It is important that floc size and distribution through the floc cells be observed daily, along with a check of mixer speed. No other control or instrumentation is necessary, other than the status normally required for the motor drives.

With hydraulic flocculation, mixing energy is a function of headloss (and thus flowrate) through the floc cells. Thus, the only means of controlling hydraulic flocculation is to vary the flowrate. Control of the process is limited to manual control of flowrate, or where applicable, manual manipulation of valves to control the flow through the process to increase/decrease retention time.

6.1.6.2.5. Clarification

Careful monitoring and control is most important to successful clarification. Adequate instrumentation to measure water quality parameters prior to and after clarification is essential.

6.1.6.2.5.1. Sedimentation

Sedimentation can be accomplished in horizontal flow, or up-flow solids contact clarifiers. The latter combines sedimentation with the chemical coagulation and flocculation processes.

For all the types mentioned, provisions should be made to observe the clarification process. These observations should focus primarily on the floc condition throughout the tanks. Poorly settling floc, a change in floc size, flow carryover onto the filters, are all indicators that the clarification process may not be optimal. Jar testing should be undertaken to determine if the coagulant (and polymer) dosage, flocculation energy or flowrate needs adjusting.

The operator should watch for floating sludge; algae growth on tank walls and launders; and any abnormal appearance in the process water.

For horizontal or up-flow clarifiers that are equipped with mechanical sludge removal, it is necessary to monitor sludge quality and consistency. For smaller plants this may be done manually by drawing daily samples and noting the concentration, texture and condition of the sludge. Provision should be made for this sampling. In larger installations, ultrasonic or magnetic sludge density meters should be provided. Sample points should be located at strategic depths in the sludge hopper to provide information on sludge accumulation rate. An alarm should be provided to indicate failure of the mechanical sludge removal equipment.

Information gathered from this type of monitoring plus the operator’s experience will be used to maintain control of the process. Provision should be made for the operator to adjust the sludge sweep cycle, speed and duration.

Up-flow solids contact clarifiers are more complex to operate because they rely on maintaining a sludge blanket to develop and capture the floc from the rising f low. Although a very efficient clarification process, it is sensitive to hydraulic and solids shock loading and should preferably be run continuously at as steady a flowrate as possible.

Starting or restarting a solids contact clarifier (SCC) requires establishing the sludge blanket. This may take some days on initial start-up, less on restarting an operational unit, depending on the sludge condition and quality.

Monitoring of the sludge blanket position, depth and condition once it is established is critical to the operation. Typically, this can be done through manual sampling from tapings at various strategic depths within the tank. Again, the use of sludge density monitors should be used in larger installations to provide continuous monitoring of and data on the sludge blankets.

With the proper level of monitoring and the operator’s experience, the SCC process variables can be controlled. These including re-circulation turbine speed, sludge scraper speed (where applicable), sludge blow down cycle (frequency, duration), and chemical feed rates. Turbine speed and sludge scraper speed are normally adjusted manually to suit operating conditions. The sludge blow down cycle can be initiated by the volume of water processed, a timer, and/or sludge level. Again, the volume, timer and/or sludge level set points will be manually set by the operator based on raw water conditions, sludge accumulation rate and sludge condition.

6.1.6.2.5.2. Dissolved Air Flotation

The process variables in DAF are:

1. Flowrate;
2. Recycle Rate; and
3. Float Removal Cycle.

The DAF process is comprised of multiple tanks operating in parallel. To handle varying flowrates, the operator should to the extent possible match the number of units operating to the demand. The process can be shut down and restarted very easily, with the effluent quality being restored quickly (within 10-20 minutes). Shutting down units during low demand keeps the operating units performing optimally, and also reduces the amount (and cost) of recycle.

Recycle rate is set based on raw water quality and operating experience. Recycle flow is controlled by the recycle pumping rate and the nozzles through which the recycle is released into the DAF tank(s). Pressure in the recycle system upstream of the nozzles should be kept in the range 450 to 725 kPa to maintain the air in solution and provide the proper micro sized bubbles on release into the tank. Recycle flowrate can therefore be varied, provided it does not cause the pressure to fall outside the design limits. Recycle rates may be further varied by shutting down bank(s) of nozzles and/or adjusting nozzles (if applicable).

The saturation process can use either injectors or a packed bed saturator. Injectors discharge saturated water into a pressure vessel. With the packed bed saturator, the saturator itself is a pressure vessel. Within this pressure vessel, the level is monitored; this level controls the recycle system.

Variable speed recycle pumps, controlled by level in the saturator tank, are used for packed bed systems. When banks of nozzles are opened/closed, the resultant recycle flowrate change initiates a pump speed adjustment to restore the level set point. Depending on the number of pumps and control system, additional recycle pumps are brought on-line or dropped off in response to saturator tank level.

Alternatively, with injector or small packed bed systems, dedicated fixed speed pumps may be used. Each injector or tank would have its own recycle pump, which would operate whenever its DAF train comes on-line.

Float removal may be continuous or intermittent, and can be accomplished mechanically and/or hydraulically. Alarms should be provided for equipment failure or if water levels fall outside of limits.

There are various systems, some proprietary, used for water clarification. The control systems for each will not be described herein. The following provides general guidance on the recommended monitoring and control for float removal, and should be read in conjunction with manufacturer's operating instructions, where applicable. Provisions must be made for the float to be observed. This is necessary to ascertain if chemical feed and/or flocculation energy need adjustment.

Continuous float removal is achieved by a mechanical sludge skimmer, or paddle, pushing float over a weir (beach). Water level has to be maintained relatively constant for the float removal system to operate efficiently. Level is controlled either by a tank effluent weir, which is set for the design flow, and should not require adjustment unless conditions change (e.g. settlement of tankage, new design flow etc.), or by an effluent control valve with a level controller. The float removal system variables (travel/rotation speed) should have the capacity for manual adjustment. Intermittent float removal typically utilizes hydraulic methods. Water level in the DAF tank is raised, by restricting the effluent flowrate, to allow float to discharge over a weir. When the float has been discharged, the effluent flowrate is restored and water surface drops to its normal operating level. The float removal cycle (frequency and duration) should have the capacity for manual adjustment.

6.1.6.2.6. Filtration

Filtration is the most critical stage in the particulate removal process. It needs to be monitored and controlled closely to ensure treated water quality is consistent and within guidelines. The following two types of filtration are used for water treatment:

1. Rapid Gravity Filtration; and
2. Slow Sand Filtration.

6.1.6.2.6.1. Rapid Gravity Filtration

The majority of municipal water filtration plants use rapid gravity filters. Rapid gravity filters (RGF) are operated in one of two ways:

1. Constant Rate; and
2. Declining Rate.

The control systems for each are different and are described in the following sub-sections.

6.1.6.2.6.1.1. Constant Rate

Flow through a constant rate RGF is controlled by a flow-control valve on the filter effluent, or by influent flow splitting and filter level control. For the flow control type, the effluent valve position is controlled by a flowrate signal from a flow meter, usually located on the filter effluent. For the level control type, the effluent valve position is controlled by the water level in the filter.

A filter run will be terminated, and the bed backwashed on one or any of the following:
1. Run time;
2. Headloss across the bed;
3. Effluent turbidity; and
4. Effluent particle count (optional).

The termination of a filter run and start of a backwash cycle can be initiated automatically or manually. Plants that are manned continuously have this option. Plants that are not manned all the time should be designed for automatic initiation, with provision for manual override; this will ensure filters are backwashed when required, even if the operator is not there.

At a minimum, both headloss and effluent turbidity should be monitored on each individual filter. Headloss is monitored by measuring the differential pressure between the effluent line (upstream of the control valve) and the top of the filter media. The pressure signal will initiate an alarm on high level, and in a non-continuously manned plant, initiate a filter backwash cycle. On high-high level, the filter should shut down until the operator has investigated the cause of the high-high alarm and/or manually initiated a backwash.

Turbidity should be monitored by an on-line turbidimeter, and there should be one for each filter. As with headloss, a high turbidity set point can initiate an alarm (and possible backwash cycle), and a high-high turbidity set point should shut down the filter to prevent poor quality water reaching the clearwell.

The turbidimeter should be located upstream of the filter-to-waste diversion point, or a second turbidimeter should be provided for filter-to-waste if the piping is separate. Filter-to-waste duration should be based on the turbidity.

Continuous recording of effluent turbidity is required in assuring filter and plant performance.

6.1.6.2.6.1.2. Declining Rate

Flow through a declining rate RGF is not directly controlled, as is the case with constant rate RGF. The rate simply decreases as the filter plugs. An effluent valve with manually adjustable stops is set to ensure the flowrate through a clean bed is not excessive. Once set, this valve will return to the set position after backwash (or after being closed for maintenance etc).

A filter run will be terminated on one or any of the following:
1. Run time;
2. Effluent flowrate;
3. Effluent turbidity; and
4. Effluent particle count (optional).

With declining rate control, all filters have the same driving head and therefore headloss is not a useful measurement. The head over the effluent weir however is an adequate measurement of filter effluent flowrate, which will decrease gradually throughout a filter run. When flowrate measurement is provided, head over the effluent weir would typically be measured by level probe or ultrasonic level sensor, calibrated for the weir characteristics to indicate flowrate. A low level (flow) set point would initiate an alarm, and a low-low level (flow) should shut down the filter. Alternatively, an in-line flowmeter may be used.

A continuous on-line turbidity monitor should be provided on each filter upstream of the filter-to-waste diversion, or a second turbidimeter should be provided for the filter-to-waste if the piping is separate. A high turbidity set point should alarm and/or initiate a backwash. A high-high set point should shut down the filter until the operator has investigated the cause of the high-high alarm and/or initiated a backwash.

Continuous recording of effluent turbidity is required in monitoring filter and plant performance.

6.1.6.2.6.1.3. Backwashing

Backwashing a filter can be initiated various ways, as discussed earlier.

A time-initiated backwash can be automatic. Smaller plants feeding smaller systems may benefit from backwashing overnight when demand is low, and the operator is not present. In such cases, a timer can be hard wired into the filter control panel to initiate the backwash, or alternatively, the time control can be programmed into the PLC.

If an automatic backwash on turbidity, headloss (constant rate), or flow (declining rate) is desired, care should be taken to consider plant demand and the effect of interrupting or reducing production to ensure that service and treatment are not compromised.

Before proceeding, the control system must confirm that a wash is permitted by checking source water volume, receiver volume, and that no other filter is washing or that no other process or production restraints exist. Once started, its cycle can be controlled manually by the operator, or automatically by timers and a sequencer in a PLC program. All timer settings must be adjustable.

The backwash cycle takes 20 to 40 minutes depending on whether it is a straight water backwash or includes surf ace wash or air scour. Backwash flow changes must be made gradually. Valve sequencing is important. Flow set points must be adjustable.

With surface wash, a typical cycle may include:
1. Draw down water level over filter;
2. Initiate surface wash;
3. Initiate backwash before surface wash ends;
4. Filter-to-waste; and
5. Return filter to service.

With air scour, the cycle may include:
1. Draw down water level over filter to approximately 50-100 mm above media;
2. Initiate air scour;
3. Combined air scour and low rate backwash on time or until level reaches within 50-75 mm of washwater waste weir;
4. Stop air scour, initiate high rate backwash;

5. Filter-to-waste until effluent turbidity is within limits; and
6. Return filter to service.

A pressure transmitter senses water level over the filter to control the opening/closing of valves and the start/stop of surface wash, air scour and low rate backwash and filter filling after high rate backwash, if required. All other control is time based.

When a filter is put back into service, water should be filtered to waste until the turbidity has dropped to a preset acceptable value. Filter to waste should be performed at approximately the same flowrate as the filter normally operates. The on-line turbidimeter monitors filter-to-waste turbidity and once the turbidity set point is reached, a signal is sent for the filter-to-waste valve to close and the filter-to-production valve to open simultaneously. Valve stroking during this changeover should be synchronized to maintain flowrate through the filter as constant as possible to avoid the turbidity spikes that can occur with a sudden change in flowrate through a filter.

Water used for backwashing must be filtered water. A backwash pump will supply backwash water typically drawing from the plant's treated water clearwell. Alternatively, backwash water can be supplied directly off the transmission main leaving the plant or from a header tank. Backwash flowrate control is critical to assure good cleaning and avoid media loss. The higher density of cold water in winter requires a lesser flowrate than the summer to achieve fluidization of the bed. This flowrate set point should therefore be adjusted seasonally to compensate for the water temperature variation. Backwash flowrate is be measured by in-line (magnetic or ultrasonic) flowmeter.

Control of the flowrate may be manual or automatic. Manual control may be used on straight backwash systems (with no air scour). Manual control requires adjustment to a throttling valve on the backwash supply until the desired flowrate, as measured by the flowmeter, is achieved. This valve should be locked in this position and adjusted seasonally to compensate for water temperature as described above. Excessive and undesirable disturbance of the media can be caused by a sudden rush of water on pump start-up or rapid valve opening. This is avoided by having a second in-line valve, typically a pilot operated globe valve, which opens slowly to bring the backwash flow gradually to the set point. Alternatively, a bypass valve that closes slowly may be used.

A backwash system that includes air scour, and consequently two backwash flowrates, is best controlled automatically by a PLC (or DCS). Backwash flowrate can be varied either by using variable speed drive on the backwash pump, using multiple pumps, or using an in-line flow control valve. For the variable speed drive and flow control valve options, a flow transmitter on the backwash supply will provide a signal to the PLC. The PLC will use this signal to control the pump speed/valve position.

For the multiple pump option, the pumps could be set up to provide the required flowrates using manually adjusted throttling valves on each pump discharge so that one pump could supply the low flow, and the other (or both in parallel) could meet high flow. As with manual control, flowrate changes should not be sudden, and an in-line pilot operated globe style valve should be used to avoid this.

6.1.6.2.6.2. Slow Sand Filtration

Although their function is the same as RGF, slow sand filters operate under different conditions. Slow sand filters (SSF) operate at much lower loading rates and rely on the formation of a thin layer, Schmutzdecke, on top of the sand medium. As well as forming a filter to remove particulates, this layer contains various microorganisms that remove bacteria and other organisms. SSF are not backwashed, rather the Schmutzdecke layer is removed occasionally (typically every 4-8 months) when headloss becomes excessive, flowrate drops off and/or quality starts to deteriorate. Because of the very slow flowrate through SSF, headloss, flowrate and effluent quality can remain very stable for many weeks.

The operator can make adjustments to the flowrate manually. When commissioned, SSFs have to mature over a period of several weeks before they can produce potable quality water. Once a filter has been brought on-line, the primary operator function is to monitor and control flowrate. Flowrate through a filter is measured either by measuring head over the effluent weir, or preferably by the more accurate method of in-line flowmeter on the filter effluent pipe upstream of the filter recycle diversion. A manual throttling valve on the effluent pipe should be adjusted to ensure flowrate through a new filter is not excessive. A manual throttling valve on the filter inlet should also be adjusted to ensure influent flowrate closely matches the effluent flowrate. These valves should be adjusted to maintain the desired water depth over the bed. Alternatively, starting and stopping the raw water pumps may control water level over the filter. As the bed clogs and headloss increases, the effluent valve should be opened to compensate.

Instrumentation should be provided to routinely monitor raw and treated water quality. A sudden increase in headloss accompanied by a reduction in flowrate signals that the filter is plugged.

6.1.6.2.7. Disinfection

Chlorine is the most common disinfectant used in potable water treatment. The following discussion provides process control guidelines for the use of chlorine in either its gaseous or solution forms.

Effectiveness of chlorine disinfection and the dosage is a function of the water temperature, pH, the contact time and the chlorine demand. The dosage is controlled on the basis of the measured residual.

Disinfection is controlled for the following reasons:

1. To meet CT requirements, which will vary with the raw water quality and the performance of the upstream processes; and
2. To ensure that chlorine residual for water leaving the plant is maintained at the set point.

Laboratory chlorine demand analysis will establish the required dose concentration. The chlorine dose rate will then be controlled by a signal from the flow transmitter on the flowmeter measuring the flow in the process where the chlorine will be injected. The chlorine residual will be monitored downstream of the process/tankage that provides the required CT. This measured variable can be used to automatically adjust the dose concentration through feedback loop control, and/or provide an alarm status to which the operator would have to respond. Unless the plant is manned continuously, it is preferable that automatic control be provided.

Since treated water is stored at the plant for some time, its chlorine residual can diminish. The residual will thus need to be 'topped up' or trimmed to the proper concentration. The trim dose is controlled by the flowrate (not the plant production rate) and chlorine residual of the water leaving the plant. Feedback loop control using a chlorine residual analyzer combined with flow-pacing control from the discharge flowmeter will automatically maintain the set point residual. An alarm should be generated if the residual falls out of bounds.

Chlorine gas feed systems incorporate a low-pressure switch on the vacuum gas line to the chlorine feeder to provide an alarm signal, and where applicable, signal an automatic switch over from duty chlorine cylinder/container to standby cylinder/container. Chlorine feed rate is controlled by modulating the chlorinator. The system is started and stopped by opening or closing the plant service water valve supplying the chlorine injector. A chlorine gas flow meter or solution flow proving switch is desirable. A chlorine gas detector must be provided in the chlorine room; the detector should be connected to a remote audible and visual alarm system.

6.1.7. 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:

1. Design and construction standards, specifications and installation details;
2. Panel sizing and general arrangement;
3. Control system functional requirements;
4. Control component and instrument data sheets;
5. Operator interface and control hardware and software specifications including input and output (I/O) lists; and
6. Control system programming and packaged system configuration standards, structure and scope.

6.1.8. Control System Documentation

The following documents should be provided following completion of the control system:

1. Record drawings to show any changes to the design and including any drawings produced during construction;
2. Annotated listings of control system programs and packaged system configuration;
3. Manufacturer's literature for all control and instrumentation components;
4. Final wiring diagrams complete with wire and terminal coding;
5. Motor control schematics;
6. Instrument loop diagrams;

7. Panel wiring and layout details;
8. PLC or DCS wiring schematics;
9. Instrument calibration sheets; and
10. Operating instructions.

6.1.9. Training

Adequate training to the plant operating and maintenance staff should be provided so that the system can be operated to meet the design criteria. Safety training for chlorine and chemical handling, including spill clean-up and first aid should be included.

6.2. Sewage Works – Instrumentation and Controls

Several factors should be considered when developing a plan for the instrumentation and controls for a wastewater treatment facility. AEP monitoring requirements vary depending on the type of facility being considered and its location; this will impact on the selection and type of instrumentation being considered. Instrumentation and control requirements will also depend on the size of the plant, and as each treatment process has its own set of conditions to be monitored and controlled there will be different technical requirements to be met. In general, instrumentation and control should provide efficient and safe automatic and manual operation of all plant systems with a minimum of operator effort. Automatic systems should also be provided with manual back-up systems.

The design should have provision for local control systems where parts of the plant may be operated or controlled from a remote location. The local control stations should include provision for preventing the operation of equipment from remote locations.

When making decisions relating to instrumentation and control, the following factors should be considered:

1. Plant size and complexity;
2. Regulatory requirements;
3. Hours of attended operation;
4. Potential chemical and energy savings;
5. Primary element reliability
6. Primary element location
7. Whether controls should be manual **or** automatic; and
8. The data storage and recording requirements and whether data acquisition should be central or distributed

For effective operation of larger wastewater treatment facilities the following parameters should be measured, some may not be required for smaller facilities.

1. Flowrate for raw sewage, by-pass flows, final effluent flow;
2. Return Activated Sludge (RAS) flows, Waste Activated Sludge (WAS) flows;
3. Raw and digested sludge flow, digester supernatant flows;
4. Chemical dosage, digester gas production;

5. Hazardous gas monitoring;
6. Anaerobic digester temperature;
7. Dissolved oxygen levels; and
8. Sludge blanket levels and sludge concentrations.

6.2.1. 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.

6.2.1.1. Flow Measurement

6.2.1.1.1. Magnetic Flowmeters (Mag Meters)

Liner and Electrode Materials **-** The liner for the meter can vary depending on the application being considered. In applications where moderate amounts of abrasion are likely to occur, one of the following materials may be selected; Polyurethane, Butyl rubber, Neoprene or Polytetrafluoroethylene. In applications where corrosion is likely to occur, one of the following materials may be selected; Ceramic or Polytetrafluoroethylene. Stainless steel electrode material should be used for applications where corrosion is not likely to present a problem. Hastelloy electrode material should be used for applications where corrosion is likely to present a problem.

Installation **-** Installation of magnetic flow meters generally require five straight pipe diameters upstream of the meter and three down stream of the meter free of valves or fittings. Meters may be installed on horizontal, vertical or sloping lines. It is essential to keep the electrodes in the horizontal plane to assure uninterrupted contact with the fluid or slurry being metered. The operating velocity required for these meters will fall into the range or (1 to 10 m/s) for non-solids bearing liquids and (1.5 to 7.5 m/s) for solids bearing liquids. 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.

6.2.1.1.2. Ultrasonic Flowmeters

Flowmeter Construction **-** The flowmeter usually consists of an electronics housing, transducers and pipe section. These can in many cases be fitted to existing pipes either by drilling holes for the transducer hardware or by application of external transducers to the outside of the pipe. When installed on existing pipes, the existing pipe material should be checked to assure it will not dampen the sonic signal as this will adversely affect performance.

Installation - The installation of Ultrasonic flow meters generally require ten to twenty straight pipe diameters upstream of the meter and five down stream 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 fall into the range of (1 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.

6.2.1.1.3. 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 ten straight and as high as fifty pipe diameters upstream of the meter and five down 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.

6.2.1.1.4. Flumes and Weirs (Parshall Flume)

Installation - The flume will be affected by upstream channel arrangement and it is recommended that there be at least ten channel widths upstream. The flume must also be installed carefully to make certain that it is level.

Applications **-** Flumes and weirs are customarily used to measure flows in open channels. They are recommended for applications involving open channel flow measurement.

6.2.1.2. 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 suspended solids concentrations less than 100 mg/L.

6.2.1.3. Suspended Solids Measurement (Optical)

Installation - Installation details for optical analyzers are unique to each manufacturer. The manufacturer's recommendations should be followed.

Applications - Optical analyzers are recommended for applications involving solids concentrations from 20-mg/L to 8%. Examples are, RAS, WAS and mixed liquor.

6.2.1.4. Dissolved Oxygen Measurement (Galvanic)

Installation - Installation details for dissolved oxygen analyzers are usually related to the choice of placement of the analyzer in the process fluid. The analyzers generally require fairly frequent maintenance and this should be considered in determining the location for installation.

Applications - Oxygen analyzers are recommended for applications involving oxygen concentrations from 0 to 20 mg/L.

6.2.1.5. Level Measurement

6.2.1.5.1. Sonic Ultrasonic

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 - This type of level element may be used in many level and flow applications; it is not recommended in locations where foam is dense and persistent.

6.2.1.5.2. Float

Installation - Float switches are normally located in a stilling well when turbulence is expected.

Applications - Float switches are commonly used for high and low level alarms and for controlling pump starts and stops.

6.2.1.5.3. Capacitance

Installation - The installation practices can vary and the manufactures recommended installation should be used.

Applications - May be used in applications that require continuous level measurement and also as switches for alarms or start/stop control.

6.2.1.6. Pressure Measurement

6.2.1.6.1. Bourdon Tubes

Installation - The installation practice should include the use of block and bleed valves.

Applications - May be used in applications that require pressure indication. Pressure range 0 to 35000 kPa.

6.2.1.6.2. Bellows

Installation - The installation practice should include the use of block and bleed valves.

Applications - May be used in applications that require pressure indication. Pressure range 0 to 2000 kPa.

6.2.1.6.3. 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 - May be used in applications that require pressure indication or transmitter output. Pressure range 0 to 3500 kPa.

6.2.1.7. Temperature Measurement

6.2.1.7.1. Thermocouples

Installation - The thermocouple should be selected with care to assure that the appropriate device is chosen for the given temperature range. Installation with a thermowell is advised.

Applications - Thermocouples are suitable for most temperature measurement applications.

6.2.1.7.2. Resistance Temperature Detector

Installation - The resistance detector should be selected with care to assure that the appropriate device is chosen for the given temperature range. Installation with a thermowell is advised.

Applications - Resistance detectors are suitable for temperature measurement applications with ranges of 0 to 300ºC.

6.2.1.7.3. Thermistor

Installation - The Thermistor should be selected to assure that the device is appropriate for the temperature range. Installation with a thermowell is advised.

Applications - Thermistors are suitable for temperature measurement applications with ranges of 0 to 300ºC.

6.2.1.7.4. Thermal Bulb

Installation - No special installation requirements.

Applications -Thermal bulbs are suitable for temperature measurement applications with ranges of 0 to 500ºC.

6.2.2. Process Controls

6.2.2.1. Lift Stations

Lift stations require simple and dependable instrumentation and control systems. The parameters that should be monitored are level, flow, pressure, temperatures, hazardous gas levels, as well as status and alarm conditions. The monitoring and control requirements will vary for each individual case based on the size, location, and economic considerations.

6.2.2.1.1. Level Control

Lift stations vary in size and storage capacity but generally they require similar controls. The level in the wet well increases to the point where a duty pump will be required to start, a lag and 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 acceptable level control devices.

When variable speed pumps are used there are several ways in which the pump can be controlled. These generally are controlled to maintain a level set point in the wetwell. 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 in order to maintain the set point value. Level control of this nature require reliable analog level measurement if it is to function properly. Regardless of the type of level control selected, the system should include a separate low level lockout and high level alarm.

6.2.2.1.2. Flow Monitoring

The flow metering element should be selected carefully to ensure that there are no obstructions where clogging may occur. Provision should be made so that the flow-metering element can be bypassed or isolated for routine maintenance activities. The flow-metering device should be connected to either 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.

6.2.2.1.3. 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 either the control system or to a recording device or both.

6.2.2.1.4. Pumps and Motors

The following parameters should be monitored:
1. Pump bearing temperature;
2. Pump bearing vibration;
3. Pump speed for variable speed applications;
4. Pump discharge pressure;
5. Motor voltage and current;
6. Motor hours of operation;
7. Motor bearing temperature; and
8. Motor windings temperature.

6.2.2.1.5. Alarms

Lift stations should be alarmed as outlined in Section 5.3.15.

6.2.2.2. Mechanical Bar Screens

Three methods are used to control the operation of mechanical bar screens:
1. Simple manual start/stop, which requires the presence of the operator at the screen in order to start and stop the screen.

2. Automatic activation by differential level. This method 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 (i.e. limit switch, proximity sensor, timer). In addition, a timer should be provided to initiate a cleaning cycle at regular intervals regardless of actual head loss. When this method is employed, there should be an alarm signal with a head loss set at a point higher than the automatic start of the mechanical bar screen.

3. Automatic activation by timer with differential level as emergency start condition. This method 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 (i.e. limit switch, proximity sensor, timer). When this method is employed there should be an alarm signal with a head loss set at a point higher than the automatic start of the mechanical bar screen.

6.2.2.3. Primary Treatment

6.2.2.3.1. Raw Sludge Pumping

The raw sludge pumping should be set up to incorporate the following features:

1. Automatic or manual selection of duty pump;
2. On line sludge density metering for control and monitoring;
3. On line sludge flow monitoring and totalization;
4. On line adjustable sludge density control;
5. Individually selectable hopper pumping controls where required;
6. Manual override for automatic controls;
7. On line sludge blanket level monitoring and alarming;
8. On line sludge pump monitoring and control;
9. Sludge density feedback control for variable speed pumping with manual override;
10. On line sludge pump speed monitoring and control with manual override; and
11. On line monitoring and control of primary tank scraper mechanisms.

6.2.2.3.2. Scum Pumping

The scum pumping should be set up to incorporate the following features:

1. Automatic or manual selection of duty pump;
2. Manual override for automatic controls;
3. On line sludge blanket level monitoring and alarming;
4. Automatic controls consisting of high and low scum tank level for starting and stopping scum pumps;
5. High scum tank level alarm;
6. On line scum pump speed monitoring and control with manual override; and
7. Scum tank flushing system for scum tank cleaning

6.2.2.4. Secondary Treatment

6.2.2.4.1. Dissolved Oxygen (DO) Control

Automatic DO control systems should be used to control the rate of air supply to aeration tanks. The following methods may be used:

1. **Closed Loop Control (Feedback Control)** - Closed loop control consists of on line dissolved oxygen analyzers providing feedback control to an airflow control device. The dissolved oxygen reading is compared to the dissolved oxygen set point. The resultant error signal is used to increase or decrease the rate of air flow to the aeration tanks. Automatic dissolved oxygen control should always be equipped with manual override.

2. **Feed Forward Control** - Feed forward control consists of a fixed volume of air being delivered to the aeration tanks for a given flowrate. This system may utilize on line dissolved oxygen analyzers but these are used for monitoring only and do not provide feedback to the air flow control elements. Process status and alarms should be provided for dissolved oxygen level, blower operating parameters, air flow control elements.

6.2.2.4.2. Return Activated Sludge Control

The Return Activated Sludge pumping should be set up to incorporate the following features:

1. Automatic or manual selection of duty pump;
2. Variable speed pumping;
3. Return activated sludge flow monitoring;
4. Feedback control to match pumping rates to flow set points;
5. Individual control of sludge return rate from individual final clarifiers;
6. Manual override for automatic controls; and
7. On line monitoring of return sludge flowrate, pump speed and status.

6.2.2.4.3. Waste Activated Sludge Control

The Waste Activated Sludge pumping should be set up to incorporate the following features:

1. Automatic or manual selection of duty pumps;
2. Variable speed pumping;
3. Waste Activated Sludge flow monitoring;
4. Feedback control to match pumping rates to flow set points;
5. Manual override for automatic controls; and
6. On line monitoring of Waste Sludge flowrate, pump speed and status.

6.2.2.4.4. Chemical Control System

Chemical addition 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 such things as return sludge flowrate. Chemical dosing requirements will vary widely depending on performance requirements and the specific process being utilized.

6.2.2.4.5. Disinfection Control Systems (Ultra Violet)

The disinfection of final plant effluent utilizing ultra violet light consists of a feed forward control system. This consists of a series of lamps and or lamp channels that are turned on based on effluent flow of the plant, UV transmittance analyzers may be utilized for monitoring system performance but are not generally employed in feedback control.

6.2.2.5. Control and 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, PLC (Programmable Logic Controller) Systems and Distributed Control Systems.

6.2.2.5.1. 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.

6.2.2.5.2. PLC Control Systems (Programmable Logic Controllers)

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 controller programs.

Plant personnel require process information in real-time or in near real-time. The PLC systems accomplish this by means of a Man Machine Interface (MMI). The MMI may be dedicated hardware and software or may come in the form of personal computers utilizing MMI 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.

6.2.3. 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:

1. Design and construction standards, specifications and installation details;
2. Panel sizing and general arrangements;
3. Control system functional requirements;
4. Control component and instrument data sheets;
5. Operator interface and control hardware and software specifications including input/output lists; and
6. Control system programming and packaged system configuration standards, structure and scope.

6.2.4. Control System Documentation

The following documents should be provided following completion of the control system:

1. Record drawings to show any changes to the design and including any drawings produced during construction;
2. Annotated listings of control system programs and packaged system configuration;
3. Manufacturer's literature for all control and instrumentation components;
4. Final wiring diagrams complete with wire and terminal coding;
5. Motor control schematics;
6. Instrument loop diagrams;
7. Panel wiring and layout details;
8. PLC or DCS wiring schematics;
9. Instrument calibration sheets; and
10. Operating instructions.

6.2.5. Training

Adequate training should be provided to the plant operating and maintenance staff so that the system can be operated to meet the design criteria.

Section 7 – Operator Training

7. Operator Training

If a water distribution, wastewater collection, water treatment or wastewater treatment system is to operate efficiently and properly, continuous and skilled attention is the most important component. Without proper attention, mechanical devices will fail and systems will cease to produce the desired results. Although modern mechanical devices have greatly reduced the amount of labour required and to a large extent have eliminated the more disagreeable operational tasks, there is no completely self-operating system.

The DOEC is responsible for ensuring that water works and sewerage works are maintained and operated in a manner that provides safe, clean drinking water for the present and future generations of Newfoundland and Labrador, as well as, providing for the protection of the natural environment.

The availability of trained and qualified operators is vitally important to the safe and sustainable operation of municipal water and wastewater systems. Operator education and training are as essential to the successful operation of these systems as are improved technologies, regulatory inspection, and monitoring. Without adequately trained personnel, the most advanced technology and regulatory compliance cannot reliably deliver safe drinking water, or provide acceptable levels of wastewater treatment. These systems will not retain their economic viability without regular maintenance by adequately trained operators.

In 2002, the DOEC made changes to the permitting process for water and sewerage systems. For new construction, modifications, or extensions to systems, proponents are required to obtain a **Permit to Construct** to address design and construction issues. The Permit to Construct replaces the previously issued Certificate of Approval. In addition to the Permit to Construct, the owners of all municipal water distribution, water treatment, sewerage collection, and sewerage treatment systems will be issued a **Permit to Operate**. The Permit to Operate will be based on the system's classification, and will stipulate general operating and system maintenance requirements for each system, as well as noting system-specific conditions where appropriate. All systems will be required to have trained operators (with operator certification strongly recommended); to keep maintenance and operational logs; and to submit annual Operation and Maintenance Reports to the DOEC. Municipalities will be required to submit the names of their system operators, and a database will be maintained regarding the education and training of individual operators. The Permit to Operate will recommend continuing education for all operators.

7.1. Training Programs and Operator Qualification

Although the certification of water and wastewater systems operators is not yet mandatory within the Province, the DOEC does make training programs available on various aspects of maintenance and operation.

The authorities responsible for plant operations should encourage operator attendance at these training programs, as well as at others that may become available. The highlights of various training and certification programs offered by the province are as follows:

1. **Educational Seminars** - Educational seminars consist of classroom training such as seminars, workshops, and certification-specific curriculum, which are presented on a regular basis. These seminars are offered at over 20 strategically located towns throughout the province, and generally do not require operators and municipal staff to travel more than 50 km. The training curriculum is based on established training manuals, which will provide operators with the knowledge to properly operate and maintain their systems, as well as to be able to write certification exams.

   This component of the program will also provide opportunities to meet the continuing education requirements for operators. The DOEC recommends a 24 hour annual continuing education training for all operators. Changing technology, regulatory requirements, and a general need for personnel in the water works industry to remain current, requires continuing education.

2. **On-Site Training** - This component of the training program is designed to raise the competency level of operators to an acceptable standard. It is site based, one-on-one, hands on, and tailored to the needs and aptitude of the operator. The training deals with proper operation/maintenance of the water supply system. The on-site training program utilizes 3 Mobile Training Units based out of St. John’s, Grand Falls-Windsor, and Corner Brook, and are manned by the Operator Trainers. These vehicles are equipped with tools, demonstration equipment, and a work area so the operators can benefit from hands-on training specific to their own system components. In addition to providing operation and maintenance training, this program augments the classroom training provided by the Operator Education component.

   On-site training is seen as a very important component of the overall training program. It is expected that the operators will respond positively to hands on training especially in the familiar surroundings of their own facility.

3. **Certification Examinations** - The DOEC will coordinate special courses for certification, administer certification exams, and maintain databases on operator training and certification. Those operators who meet the basic educational and experiential qualifications established by the *ACWWVCB*, may write the appropriate level of certification exam and become certified if they pass the *ABC* exams with a mark of 70% or greater. Those operators who attend all of the educational seminars in their area but do not wish to write the certification exam, or those who do not pass the exam, will be issued a *Certificate of Participation* by the DOEC, in recognition of their efforts.

7.2. Operating Manual

The purpose of an Operating Manual is to provide an understanding of the processes and general operation of water and sewerage systems. This type of manual is not intended to indicate every single step in the operating procedure or to attempt to replace the Suppliers' Equipment Manuals.

An Operating Manual should provide a basic understanding of what the system is supposed to accomplish; the philosophy of the design; specific criteria for satisfactory operation and the identification of potential operational problems.

The Operating Manual should contain, where pertinent, at least the following items in sufficient detail:

1. Purpose of manual;
2. Terminology and definitions;
3. General description of water and sewerage systems;
4. Description of key components and their operation;
5. Operational duties (general);
6. Initial start-up procedures;
7. Normal start-up procedures;
8. Operational procedures as follows:
    a) Plant running normally;
    b) Emergency (including power failure);
    c) Maintenance; and
    d) Keeping operating records;
9. Shutdown procedures;
10. Operation safety;
11. Housekeeping;
12. Test procedures and standards;
13. List of major equipment;
14. List of interlocked equipment;
15. List of pumps;
16. List of motors;
17. List of gates and valves for the process;
18. List of construction drawings;
19. References; and
20. Miscellaneous functional drawings.

Section 8 – Occupational Health and Safety

8. Occupational Health and Safety

The design, construction, operation and maintenance of all water and sewerage systems shall incorporate all necessary measures to assure a safe working environment. The safe working environment measures should be in line with the provisions of the Occupational Health and Safety Regulations.

8.1. Fencing

Adequate fencing shall be provided around all works to prevent trespassing and vandalism.

8.2. Stairways And Handrails

Stairways should be provided in areas where daily access is required for operation and maintenance.

Handrails should be provided to ensure complete safety of operation.

8.3. Working Area

Sufficient working area shall be provided around all machinery, valves, etc. to permit ease of operation and maintenance.

8.4. Grating

Approved non-slip grating shall be used for all outdoor walkways.

8.5. Piping

Piping shall be located to provide sufficient head clearance.

8.6. Non-Potable Water

When non-potable water is made available to any part of the plant for various purposes, all yard hydrants and outlets shall be clearly marked. The appropriate colour coding shall identify all underground and exposed piping.

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Definition of Terms

"Applicant" means a municipality, utility district, authority, corporation or person for whom the water supply improvement work is to be designed, constructed or operated.

"BOD" means the total 5-day biochemical oxygen demand, which is the amount of oxygen required to stabilize biodegradable organic matter under aerobic conditions within a five-day period in accordance with the latest edition of Standard Methods.

"Combined Sewer System" means a sewerage system which carries both sanitary sewage and/or industrial wastes and storm water or drainage.

"CT disinfection" means the product of the residual disinfectant concentration (C) and the contact time (T).

"Dissolved Oxygen" means a measure of the free oxygen in water or effluent.

"Effluent" means any liquid discharged from any unit of wastewater treatment works, or from a sewer.

"Engineer" means the professional person or firm, which designed the sewage and/or water, works and conceived, developed, executed or supervised the preparation of the plan documents.

"Permit to Construct" means a permit issued by the Minister under *Section 36(1) or 37(1)* of the Water Resources *Act*.

"Permit to Operate" means a permit issued by the Minister under Section 38(1) or 38(2) of the *Water Resources Act*.

"Pollution" means any alteration of the physical, chemical, biological, or aesthetic properties of the air, soil or waters of the province, including change of temperature, taste or odour, or the addition of any liquid, solid, radioactive, gaseous, or other substance to the air, soil or waters which will render, or is likely to render the air, soil or waters of the province harmful to the public health, safety, or welfare, or harmful, or less useful for domestic, agricultural, industrial, power, municipal, navigational, recreational, or other lawful uses, or for animals, birds or aquatic life.

"Primary Treatment" means treatment for the physical removal of settleable or floatable materials from a wastewater. Primary treatment of a wastewater may also include adjustment of pH and equalization of flow.

"Secondary Treatment" means a method of waste treatment beyond primary treatment where pollutants in solution or in the colloidal state are biologically or chemically removed. The minimum treatment required under this method shall be the removal of at least 85% of the BOD and suspended solids, unless intermediate treatment is deemed to be permissible by the Department.

"Suspended Solids (SS)" means substances such as erosion silt, organic and tritus, plankton, and sand, which are held in suspension in water.

List of Acronyms

AEP – Association of Environmental Professionals
ANSI – American National Standards Institute
ASCE – American Society of Civil Engineers
ASME – American Society of Mechanical Engineers
ASTM – American Society for Testing and Materials
AWWA – American Water Works Association
CSA – Canadian Standards Association
DCS – distributed control systems
DNA – deoxyribonucleic acid
DO – dissolved oxygen
GCDWQ – *Guidelines for Canadian Drinking Water Quality*
GAC – granular activated carbon
HAA5 – haloacetic acid
HRT – hydraulic retention time
IAO – Insurance Advisory Organization
MAC – maximum acceptable concentration
MCL – maximum contaminant level
MF – micro filtration
MMI – Man Machine Interface
NSF – National Science Foundation
NTU – nephelometric turbidity units
OHSA *– Occupational Health and Safety Act*
PLC – Programmable Logic Controllers
PTA – packed tower aeration
RNA – ribonucleic acid
SCADA – Supervisory Control and Data Acquisition
SDDS – small diameter distribution system
STEP – septic tank effluent pump
THM - trihalomethane
UF – ultra filtration
USEPA – United States Environmental Protection Agency
UV – ultraviolet
WAS – waste activated sludge