Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems

Part 4
Wastewater Systems Guidelines for Design, Operating and Monitoring
of a Total of 5 Parts

March 2013

Alberta Government

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Edmonton, Alberta T5K 2P7
Tel: 780-427-4952
Fax: 780-452-0668

More information regarding "Part 4 – Wastewater Systems Guidelines for Design, Operating and Monitoring" may be obtained by contacting the document owner:

Bijan Aidun
Regional Integration Branch
Operations Division
Alberta Environment and Sustainable Resource Development
4th floor, Twin Atria
4999-98 Avenue
Edmonton, Alberta, T6B 2X3

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Part 4 WASTEWATER SYSTEMS GUIDELINES FOR DESIGN, OPERATING, AND MONITORING
March 2013

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Additional Parts published separately are:

Part 1 Standards for Municipal Waterworks
Part 2 Guidelines for Municipal Waterworks
Part 3 Wastewater Systems Standards for Performance and Design
Part 5 Stormwater Management Guidelines

FOREWORD TO PART 4 WASTEWATER SYSTEMS GUIDELINES FOR DESIGN, OPERATING AND MONITORING (2013)

Alberta Environment and Sustainable Resource Development (AESRD) has the regulatory mandate, in accordance with the Environmental Protection and Enhancement Act and Regulations, for the Drinking Water, Wastewater and Storm Drainage serving large public systems in Alberta. AESRD considers the establishment of standards and guidelines for municipal waterworks, wastewater and storm drainage facilities an integral part of our regulatory program directed at ensuring public health and environmental protection. AESRD's objective is to develop comprehensive and scientifically defensible standards and guidelines that are effective, reliable, achievable and economically affordable.

Since publication of the last revision of the Standards and Guidelines, Alberta Environment and Sustainable Resource Development has embarked on a process of "decoupling" the various components of the January 2006 document into functionally-associated sections to aid those using the document. This process started with the publication of the January 2006 version of the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems in the Alberta Gazette. A program of separating the component parts of this document is under way and new parts will eventually replace the corresponding sections in the January 2006 Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems. Until the process of "decoupling" is completed with new "Parts" the existing sections of the 2006 Standards and Guidelines document will remain in operation. This Part (Part 4) details system components that are guidance to best practices in providing well managed wastewater systems and should be read in conjunction with Part 3 – Wastewater Systems Standards for Performance and Design (2013).

The system owners / utilities are responsible for meeting AESRD's regulatory requirements and for the collection, treatment and disposal or use of wastewater. They are also responsible for maintaining the wastewater collection system up to the service connection. Engineering consultants and / or the system owners / utilities are responsible for the detailed project design and satisfactory construction and operation of the waterworks and wastewater systems.

In accordance with the Wastewater and Storm Drainage Regulation (119/1993) a wastewater system and storm drainage will be designed so that it meets, as a minimum, the performance standards and design requirements set out in the Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems, published by AESRD, as amended or replaced from time to time, or, any other standards and design requirements specified by the Regional Director. AESRD last revised its Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems in January 2006.

This document entitled Part 4, is intended to provide general guidance on how to achieve a certain level of wastewater system performance or reliability. Good engineering and best management practices are included in this Part. These are not mandatory requirements but they establish what is expected when the system owner / utility owners applies for an approval.

This version of Part 4 – Wastewater System Guidelines for Design, Operation and Monitoring includes Sections 5 and 7 of the January 2006 Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems with some corrections and the addition of Section 4.5 outlining guidelines for septage management systems.

DEFINITIONS / ABBREVIATIONS

AO	-	Aesthetic Objectives
AESRD	-	Alberta Environment and Sustainable Resource Development
AWWA	-	American Water Works Association
BDOC	-	Biodegradable Dissolved Organic Carbon
BNR	-	Biological Nutrient Removal
BPJ	-	Best Professional Judgement
BPR	-	Biological Phosphorus Removal
BPT	-	Best Practicable Technology
CBOD	-	Carbonaceous Biochemical Oxygen Demand at 5 days and 20 ^{o}C
CFID	-	Continuous feed and intermittent discharge
DAF	-	Dissolved Air Flotation
DBP	-	Disinfection By-product
DCS	-	Distributed Control System
DO	-	Dissolved Oxygen
DOC	-	Dissolved Organic Carbon
DWSP	-	Drinking Water Safety Plan
EPEA	-	Environmental Protection and Enhancement Act
F/M	-	Food to Microorganism ratio
G	-	Velocity Gradient
GCDWQ	-	Guidelines for Canadian Drinking Water Quality
GWUDI	-	Groundwater under the direct influence of surface water
HPC	-	Heterotrophic Plate Count
HRT	-	Hydraulic Retention Time
IFID	-	Intermittent feed and intermittent discharge
MAC	-	Maximum Acceptable Concentration
MLSS	-	Mixed Liquor Suspended Solids
NH_3-N	-	Ammonia nitrogen
NSF	-	National Sanitation Foundation
NTU	-	Nephelometric Turbidity Unit
ORP	-	Oxidation Reduction Potential
OU	-	Odour Unit
PLC	-	Programmable Logic Controllers
QA/QC	-	Quality Assurance / Quality Control
RBC	-	Rotating Biological Contactor
SAR	-	Sodium Adsorption Ratio
SBR	-	Sequencing Batch Reactor
SRT	-	Sludge Retention Time
TBOD	-	Total Biochemical Oxygen Demand at 5 days and 20 ^{o}C
TOC	-	Total Organic Carbon
TP	-	Total Phosphorus
TSS	-	Total Suspended Solids
TTHM	-	Total Trihalomethanes
UC	-	Uniformity Coefficient
USEPA	-	United States Environmental Protection Agency
UV	-	Ultraviolet
WHO	-	World Health Organization

Average daily design flow (water and wastewater) - The product of the following:

- design population of the facility, and
- the greatest annual average per capita daily flow which is estimated to occur during the design life of the facility.

Co-op - An organization formed by the individual lot owners served by a waterworks system, wastewater system or storm drainage system.

Granular filter media:

1. Effective Size (D_{10}) - Size of opening that will just pass 10% of representative sample of the granular filter media.
2. Uniformity Coefficient - A ratio of the size opening that will just pass 60% of the sample divided by the opening that will just pass 10% of the sample.

Groundwater - All water under the surface of the ground.

Maximum daily design flow (water) - Maximum three consecutive day average of past-recorded flows, times the design population of the facility. If past records are not available, then 1.8 to 2.0 times the average daily design flow.

Maximum hourly design flow (water) - 2.0 to 5.0 times the maximum daily design flow depending on the design population.

Maximum monthly average daily design flow (wastewater) - The product of the following:

1. design population of the facility, and
2. the greatest monthly average per capita daily flow which is estimated to occur during the design life of the facility.

Owners - Owners of the waterworks or wastewater systems as defined in the regulations.

Peak demand design flow (water) - the maximum daily design flow plus the fire flow.

Peak wastewater design flow (wastewater) - 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 (see Section 4.1.1).

Potable water – As defined in the EPEA. Other domestic purposes in the EPEA definition include water used for personal hygiene, e.g. bathing, showering, washing, etc.

Septage – The liquid, solid or semisolid material removed from septic tanks, portable toilets, and holding tanks that receive sewage from domestic sources. This does not include wastes from grease traps, industrial or commercial processes.

Sodium adsorption ratio - A ratio of available sodium, calcium and magnesium in the soil solution which can be used to indicate whether or not the accumulation of sodium in the soil exchange complex will lead to a degradation of soil structure.

$$SAR = \frac{Na}{\left[\frac{Ca}{2} + \frac{Mg}{2}\right]^{1/2}}$$

Note : All concentrations expressed in milliequivalents per litre

Surface water - Water in a watercourse.

Watercourse - As defined in the EPEA.

4.0 Wastewater Systems - Guidelines

4.1 Design Criteria

4.1.1 Estimating Wastewater Flows

The following sections outline methodologies for quantifying wastewater flows. From 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.

4.1.1.1 Residential (Population-Generated)

If no existing data exists, the peak (population-generated) flow for a residential population may be determined by the following formula:

$$Q_{PDW} = \frac{G \ x \ P \ x \ Pf}{86.4}$$

where: Q_{PDW} = the peak dry weather design flow rate (L/s)

G = the per capita average daily design flow (L/d)

P = the design contributing population in thousands

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 = $1 + 14/(4 + P^{1/2})$

where:

P = the design contributing population in thousands.

4.1.1.2 Commercial / Institutional and Industrial

1. Determination of Average Flow

   For detailed system design, the average wastewater flow from commercial / institutional and industrial land use areas is to be estimated as set out in Table 4.1 or by actual documented usage.

TABLE 4.1
EXPECTED VOLUME OF SEWAGE PER DAY*

Place	Estimated Sewage Flow Litres (gallons) Per Day
Assembly Halls	32 (7) per seat
Campsite	80 (18) per campsite
Churches	23 (5) per seat
with kitchen	32 (7) per seat
Construction Camps	225 (50) per person
Day Care Centre	113 (25) per child
Dwellings	675 (150) per bedroom
Golf Clubs	45 (10) per member
with bar and restaurant add	113 (25) per seat
Hospital	
(no resident personnel)	900 (200) per bed
Industrial and Commercial Buildings	
(does not include process water or cafeteria)	45 (10) per employee
(with showers)	90 (20) per employee
Institutions	
(resident)	450 (100) per resident
Laundries	
(coin operated)	1800 (400) per machine
Liquor Licence Establishments	113 (25) per seat
Mobile Home Parks	1350 (300) per space
Motels / Hotels	90 (20) per single bed
Nursing and Rest Homes	450 (100) per resident
Office Buildings	90 (20) per employee
Recreational Vehicle Park	180 (40) per space
Restaurants	
24-Hour	225 (50) per seat
Not 24-Hour	160 (35) per seat

Table 4.1 continued

Place	Estimated Sewage Flow Litres (gallons) Per Day
Schools	
Elementary	70 (15) per student
Junior High	70 (15) per student
High School	90 (20) per student
Boarding	290 (65) per student
Service Stations	
(exclusive of cafe)	560 (125) per fuel outlet
Swimming Pools (Public)	
based on design bathing load	23 (5) per person

* Reproduced from the Alberta Private Sewage Treatment and Disposal Regulations, Table 8.5.B.

2. Average Flow Generation Estimates for Planning

For system planning purposes, when specific land uses and zoning are unknown and the requirements of 4.1.1.2 (1) 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:

a. Commercial and Institutional Land Uses

The lower limit for Average Flow Generation should be 40 m^3/day/ha (0.46 L/s/ha).

b. 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:

$$Pf = 6.659\,(Q_{AVG}^{-0.168})$$

Following from this, the peak dry weather flow rate (Q_{PDW} in L/s) may be determined as follows:

$$Q_{PDW} = Pf.Q_{AVG}$$
$$6.659\,(Q_{AVG}^{0.832})$$

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.

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 4.1.1.1, and is to be included in the determination of the total generation for the development.

4.1.1.3 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.

4.1.1.4 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.

4.1.2 System Capacity

In general, sewer capacities should be designed for the estimated ultimate tributary population, except in considering parts of the systems that can be readily increased in capacity. For example, the wastewater treatment plant should be designed for a minimum period of 10 years with provision for expansion to handle a 20 or 25-year design flow. Outfall structures, which have high base construction costs, should be designed for the entire design horizon which is usually about 20 to 25 years. The decision is best made based on economic analysis and cost return.

4.1.3 Wastewater Collection and Treatment System

Wastewater collection system including the pumping stations should be designed for peak wastewater design flows.

Aerated lagoon systems should be designed for maximum monthly average daily design flows, with sufficient aeration to maintain a uniform solids concentration in the complete mix cell.

Mechanical wastewater treatment plants should be hydraulically capable of handling the anticipated peak wastewater design flow rates without overtopping channels and / or tankage. From a process point-of-view, however, the design of various components of the plant should be based on the following:

Screening / Grit Removal - Peak wastewater design flow rate.

Primary Sedimentation - Average design flow rates or peak wastewater design flow rate.

Aeration - Maximum monthly average CBOD loading rate in the design year is usually sufficient with predominantly domestic wastes, but the presence of significant industrial waste loadings may create sufficient diurnal variations to warrant consideration. Seasonal variations in domestic and / or industrial CBOD loading rates should also be taken into consideration. Except for short retention treatment systems such as contact stabilization or high rate processes, hydraulic retention time is seldom critical.

Secondary Sedimentation - Peak wastewater design flow rates or peak solids loading rate.

Disinfection System - Peak wastewater design flow rates.

Effluent Filtration - Peak wastewater design flow rates.

4.1.4 Sewer Outfall

Sewer outfall should be designed for peak wastewater design flow rates.

The proper siting and design of the sewer outfall is important in minimizing the impact on receiving water quality. Outfalls should be designed and located so as to obtain the greatest possible dilution of the effluent as quickly as possible during low flow periods.

Dilution is a product of initial mixing of the effluent with surrounding water and subsequent dispersion due to water movement. Initial mixing is enhanced by extending the outfall away from the shore into deeper water and often by incorporating a multiport diffuser to spread the discharge over a larger area and to increase turbulent mixing. Similarly, dispersion is aided by maximizing the separation of the discharged plume from boundary effects of the shoreline or streambed.

4.2 Wastewater Collection

4.2.1 Sewers

4.2.1.1 Materials

The material selected should be adapted to local conditions, such as: character of wastes, possibility of septicity, soil characteristics, exceptionally heavy external loadings, abrasion, hydrogen sulphide corrosion, and similar problems.

Suitable couplings shall be used for joining dissimilar materials.

All sewers should be designed to prevent damage from superimposed live, dead, and frost induced loads. Proper allowance for loads on the sewer should 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 should be used to withstand anticipated potential superimposed loading or loss of trench wall stability.

For application in which the wastewater is conveyed under pressure, or in special cases involving excessive surcharge such as inverted siphons, pressure pipes should be used. Pipe and joints should be equal to watermain strength materials suitable for design conditions.

4.2.1.2 Sizing of Sewers

It is normal practice to design sanitary sewers to have a hydraulic capacity such that the sewer is flowing at no more than 80% of the depth when conveying the estimated design peak flow. This is because the maximum velocity is achieved when the flow is at about 0.8 of depth (note: maximum flow occurs when the pipe is flowing at about 0.93 of depth. The reason for this is that as a section approaches full flow, the additional friction resistance caused by the crown of the pipe has a greater effect than the added cross sectional area).

Flow rate at a depth of 80% of the sewer diameter is approximately 86% of the sewer full capacity. Therefore, the required flow capacity for sizing of the sewer is computed using the following relationship:

$$\textit{Required sewer capacity} = \frac{\textit{Estimated design flow}}{0.86}$$

Manning equation is generally used in sizing the sewers:

$$Q = \frac{1.00}{n} AR^{2/3} S^{1/2}$$

where:

where:	Q	=	Quantity of flow (m^3/s)
	n	=	Roughness coefficient (common value used is 0.013; lower value may be used for PVC pipes based on manufacture's recommendation)
	A	=	Cross sectional area of flow (m^2)
	R	=	Hydraulic Radius (m)
	S	=	Slope (m/m).

4.2.1.3 Changes in Pipe Size

When a smaller sewer joins a large one, the invert of the larger sewer should be lowered sufficiently to maintain the energy gradient. An approximate method for securing these results is to place the 0.8 depth point of both sewers at the same elevation.

4.2.1.4 Location

At the discretion of the municipality, the sewers may be located on the sides of the undeveloped road allowances or on the verges of developed roads.

4.2.1.5 Pressure Testing

Testing of sewers is recommended when high water table is expected or encountered.

The infiltration / exfiltration rate for PVC sewer pipes and fittings may not exceed 4.6 litres per mm diameter of pipe per km length per day. Low-pressure air testing may be permitted to verify this joint tightness when tested to a maximum rate of air loss of 0.0015 ft^3 per minute per ft^2 of internal surface. Test methods to the requirements of Uni-Bell Standard UNI-B-6-90; see Appendix 1-E for test time calculation.

4.2.2 Manholes

Manholes should be durable structures for the purpose of providing convenient access to sewers for observations, inspections, flow monitoring and maintenance operations, at the same time causing a minimum of interference in the hydraulics of the sewer system.

Where corrosion is expected because of either sulphate attack or sulphide, consideration shall be given to the provision of corrosion resistant material or effective protective linings.

4.2.2.1 Location

Manholes should be installed:

- at the end of each line;
- at all changes in grade, size, or alignment;
- at all intersections; and at distances not greater than 120 m for sewers 375 mm or less; or
- 150 m for sewers 450 mm to 750 mm.

However, the limits may be exceeded if suitable modern cleaning equipment is available to handle the larger spacing.

Greater spacing may be allowed in sewers larger than 750 mm.

Cleanouts may be used only for special conditions and may not be substituted for manholes nor installed at the end of laterals greater than 50 m in length.

Manholes should not be located in areas subject to ponding during rainstorms and snowmelt.

4.2.2.2 Sizing

For sewers up to 1050 mm in size, manholes should be constructed with a diameter of at least 1200 mm. For sewers larger than 1050 mm, special type manholes or tee riser manholes may be used. Safety and entry requirements should also be considered when sizing manholes.

4.2.2.3 Drop Manholes

Drop manholes should be used when invert levels of inlet and outlet sewers differ by 600 mm or more. Where the difference in elevation is less than 600 mm, the 0.8 depth point of both sewers should be matched.

Drop manholes should be constructed with an outside drop connection. Inside drop connections (when necessary) shall be secured to the interior wall of the manhole and provide access for cleaning. Drop should be designed to minimize the release of sewer gases.

Due to the 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.

4.2.2.4 Channelling and Benching

Good design practice should prevent the depth of flow from being above the sidewalls of the manhole channelling at all times. Therefore, manhole channels should be a smooth continuation of the incoming pipe, the channel height being one-half the pipe diameter for small sewers or three-fourths the pipe diameter for large sewers (375 mm or larger).

Manhole benching should ensure both good footing for workmen and adequate space for minor tools and equipment. Benching should have enough slope for drainage, however to provide safe footing the slope should not exceed 80 mm/m.

No lateral sewer, service connection, or drop manhole pipe should discharge onto the surface of the benching.

4.2.2.5 Frame and Cover

Manhole covers should be designed having the following:

1. Adequate strength to support superimposed loads. Frames and covers are usually cast iron, however lighter weight materials may be used where there is no danger of subjection to heavy loads;

2. Adequate size to facilitate access of equipment and people;

3. A good fit between cover and frame to prevent rattling in traffic;

4. Water tightness between cover and frame to reduce infiltration;

5. Provision for ease of opening (usually a pick notch to pry the cover loose) and an additional pick hole near the edge of the cover;

6. Provision of vent holes; and

7. Resistance to unauthorized entry. The principle defence against a manhole cover being lifted by children is its weight, however during infrequent storm events it is possible that surcharge and lifting of the cover can occur. Therefore, provision should be made in the

design to eliminate the possibility of a person falling into the manhole if the cover has been dislodged.

4.2.2.6 Steps

Manhole steps should be either aluminum or galvanized steel, being wide enough to place both feet on one step. Spacing of steps should be 300 to 400 mm.

To reduce the possibility of feet slipping on manhole steps, the safety-drop type of steps are recommended. For those manholes located within a roadway, and where possible, steps should be aligned so that the person exiting from the manhole should do so facing towards oncoming traffic.

4.2.3 Inverted Siphons

Inverted siphons should have not less than two barrels, with a minimum pipe size of 100 mm. They should be provided with necessary appurtenances for maintenance, convenient flushing, and cleaning equipment. The inlet and discharge structures should have adequate clearances for cleaning equipment, inspection, and flushing. Design should provide sufficient head and appropriate pipe sizes to secure velocities of at least 1 m/s for average design flows. The inlet and outlet details should be so arranged that the 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.

4.2.4 Wastewater Pump Station

4.2.4.1 General

Wastewater pump stations in general use fall into four types:

1. wet well / dry well;
2. submersible;
3. suction lift; and
4. screw pump.

Once the need for a pump station has been determined, the designer should select the type and location that offers a proper balance between the technical needs, economics, and the environment.

Special consideration should be given to the location of the structure relative to neighbouring development in order to minimize the possible effects of noise and odour.

All weather vehicular access should be provided to all pump stations. Security fencing and access hatches with locks should also be provided.

4.2.4.2 Wet Well / Dry Well Pump Station

1. Structures

   Safety ventilation, well separation, access and safety requirements shall be in accordance with the details outlined in Section 3.3.2.

   a. Equipment Removal

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

   b. Buoyancy

      Where high groundwater conditions are expected, buoyancy of the wastewater pumping station structures should be considered and, if necessary, adequate provisions should be made for protection.

2. Pumps

   a. Protection Against Clogging

      Pumps handling wastewater from 750 mm or larger diameter sewers should be preceded by readily accessible bar racks to protect the pumps from clogging or damage. Bar racks should have clear openings as provided in Section 4.3.1.4. Where a bar rack is provided, a mechanical hoist should also be provided.

   b. Pump Openings

      Pumps handling raw wastewater should be capable of passing particles of at least 75 mm in diameter. Pump suction and discharge openings should be at least 100 mm in diameter.

   c. Priming

      The pump should be so placed that under normal operating conditions it will operate under a positive suction head, except as specified in Section 4.2.4.3.

   d. Electrical Equipment

      Electrical systems and components (e.g. motors, lights, cables, conduits, switchboxes, control circuits, etc.) in raw wastewater wet wells, or in enclosed or partially enclosed spaces where hazardous concentrations of flammable gases or vapours may be present, should comply with the Canadian Electrical Code requirements for Class I Group D, Division 1 locations. In addition, equipment located in the wet well should be suitable for use under corrosive conditions. Each flexible cable should be provided with a watertight seal and separate strain relief. A fused disconnect switch located above ground should be provided for the main power feed for all pumping stations. When such equipment is exposed to weather, it should be weather proofed. A 110-volt power receptacle to facilitate maintenance should be provided inside the control panel for lift stations that have control panels outdoors. Ground fault interruption protection should be provided for all outdoor outlets.

e. Intake

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

f. Dry Well Dewatering

A sump pump equipped with dual check valves should be provided in the dry well to remove leakage or drainage with discharge above the maximum high water level of the wet well. 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 should be sized to remove the maximum pump seal water discharge which would occur in the event of a pump seal failure.

g. Pumping Rates

The pumps and controls of main pumping stations should be selected to operate at varying delivery rates. Insofar as is practicable, such stations should be designed to deliver as uniform a flow as practicable in order to minimize hydraulic surges. The design flow should be adequate to maintain a minimum velocity of 0.6 m/s in the forcemain.

3. Valves

a. Suction Line

Suitable shut-off valves should be placed on the suction line of dry pit pumps.

b. Discharge Line

Suitable shut-off and check valves should be placed on the discharge line of each pump (except on screw pumps). The check valve should be located between the shut-off valve and the pump. Check valves should 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 should be capable of withstanding normal pressure and water hammer.

All shut-off and check valves should be operable from the floor level and accessible for maintenance. Outside levers are recommended on swing check valves.

4. Wet Wells

a. Divided Wells

Where continuity of pumping station operation is critical, consideration should be given to dividing the wet well into two sections, properly interconnected, to facilitate repairs and cleaning.

b. Size

The design fill time and minimum pump cycle time should be considered in sizing the wet well. The effective volume of the wet well should be based on design average flow and a filling time not to exceed 30 minutes unless the facility is designed to provide flow equalization / storage. The pump manufacturer's duty cycle recommendations may be utilized in selecting the minimum cycle time. When the anticipated initial flow to the pumping station is less than the design average flow, provisions should be made so that the fill time indicated is not exceeded for initial flows. When the wet well is designed for flow equalization as part of a treatment plant, provisions should be made to prevent septicity. The well and the pumps should also be configured to avoid settlement of solids in the wet well.

c. Floor Slope

The wet well floor should have a minimum slope of one-to-one to the hopper bottom. The horizontal area of the hopper bottom should be no greater than necessary for proper installation and function of the inlet.

d. Air Displacement

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

5. Flow Measurement

Suitable devices for measuring wastewater flow should be provided at all pumping stations. Indicating, totalizing and recording flow measurement devices / instruments should be provided at large pumping stations with peak design flow greater than 50 L/s. Elapsed time meters may be used for pump stations with peak design flow less than 50 L/s.

4.2.4.3 Suction-Lift Pump Station

Pump Priming and Lift Requirements

Suction-lift pumps should be of the self-priming type and should meet the applicable requirements of Section 4.2.4.2. Suction-lift pump stations using dynamic suction lifts may exceed the limits outlined in the following sections if the manufacturer certifies pump performance and submits detailed calculations indicating satisfactory performance under the proposed operating conditions. Such detailed calculations should include static suction-lift as measured from "lead pump off" elevation to centre line of pump suction, friction, and other hydraulic losses of the suction piping, vapour pressure of the liquid, altitude correction, required net positive suction head, and a safety factor of at least 1.8 m.

The pump equipment compartment should be above grade or offset and shall be effectively isolated from the wet well to prevent the humid and corrosive sewer atmosphere from entering the equipment compartment.

1. Self-Priming Pumps

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

2. Vacuum-Priming Pumps

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

4.2.4.4 Submersible Pump Stations

Submersible pump stations should meet the applicable requirements under Section 4.2.4.2, except as modified in this Section.

1. Construction

   Submersible pumps and motors should be designed specifically for raw wastewater use, including totally submerged operation during a portion of each pumping cycle and should meet the requirements of the Canadian Electrical Code for such units. An effective method to detect shaft seal failure or potential seal failure should be provided.

2. Pump Removal

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

3. Electrical

   a. Power Supply and Control

      Electrical supply, control, and alarm circuits should be designed to provide strain relief and to allow disconnection from outside the wet well. Terminals and connectors should be protected from corrosion by location outside the wet well or through use of watertight seals. If located outside, weatherproof equipment should be used.

   b. Controls

      The motor control centre should 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 should be so located that the motor may be removed and electrically disconnected without disturbing the seal.

c. Power Cord

Pump motor power cords should be designed for flexibility and serviceability of the Canadian Electrical Code standards for flexible cords in wastewater pump stations. Ground fault interruption protection should be used to de-energize the circuit in the event of any failure in the electrical integrity of the cable. Power cord terminal fittings should be corrosion-resistant and constructed in a manner to prevent the entry of moisture into the cable, should be provided with strain relief appurtenances, and should be designed to facilitate field connecting.

4. Valves

Valves required under Section 4.2.4.2 (4) should be located in a separate valve pit. Valve pits may be dewatered to the wet well through a valved drain line. 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 4.2.4.4 (2).

4.2.4.5 Alarm Systems

Alarm systems should be provided for pump stations. The alarm should be activated in cases of high and low liquid levels, power failure, sump pump failure, pump failure, unauthorized entry, or any cause of pump station malfunction. Pumping station alarms should 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 should 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 installed in some cases in lieu of the telemetering system outlined above, depending upon location, station holding capacity and inspection frequency.

4.2.4.6 Emergency Operation

1. Objective

Wastewater pumping stations should be designed and operated in such a way that equipment breakdown may not result in the discharge of raw or partially treated wastewater to any waters and to protect public health by preventing back-up of wastewater and subsequent discharge to basements, streets, and other public and private property.

2. Emergency Pumping Capability

Emergency pumping capability should be included unless on-system overflow prevention is provided by adequate storage capacity. Emergency pumping capability may be accomplished by connection of the station to at least two independent power grids, or by provision of portable or in-place internal combustion engine equipment which will generate electrical or mechanical energy, or by the provision of portable pumping equipment. Such emergency standby systems should have sufficient capacity to start up and maintain the total related running capacity of the station. Regardless of the type of emergency standby system provided, a riser from the forcemain with rapid connection capabilities and appropriate valving should be provided for lift stations to hook up portable pumps.

3. Emergency High Level Overflows

For use during possible periods of extensive power outages, mandatory power reductions, or uncontrollable emergency conditions, 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 wastewater 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 should also be given to the installation of storage / detention tanks, or basins, which should be made to drain to the station wet well. Overflows should be considered only in conjunction with emergency pumping capability as outlined in Section 4.2.4.6 (2).

4. Equipment Requirements

a. General

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

i. Engine Protection

The engine must be protected from operating conditions that would result in damage to equipment. Unless continuous manual supervision is planned, protective equipment should be capable of shutting down the engine and activating an alarm on site and as provided in Section 4.2.4.5. Protective equipment should monitor for conditions of low oil pressure and overheating, except that oil pressure monitoring will not be required for engines with splash lubrication.

ii. Size

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

iii. Fuel Type

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

iv. Engine Ventilation

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

v. Routine Start-Up

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

vi. Protection of Equipment

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

b. Engine-Driven Pumping Equipment

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

i. Pumping Capacity

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

ii. Operation

The engine and pump should be equipped to provide automatic start-up and operation of pumping equipment unless manual start-up and operation is justified. Provisions should also be made for manual start-up. Where manual start-up and operation is justified, storage capacity and alarm system must meet the requirements of Section 4.2.4.6 (4) (ii) (c).

iii. Portable Pumping Equipment

Where part or all the engine-driven pumping equipment is portable, sufficient storage capacity with alarm system should be provided to allow time for detection of pump station failure and transportation and hook-up of the portable equipment.

c. Engine-Driven Generating Equipment

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

i. Generating Capacity

- Generating unit size should be adequate to provide power for pump motor starting current and for lighting, ventilation, and other auxiliary equipment necessary for safety and proper operation of the lift station.

- Special sequencing controls should be provided to start pump motors unless the generating equipment has capacity to start all pumps simultaneously with auxiliary equipment operating.

ii. Operation

Provisions should be made for automatic and manual start-up and load transfer unless only manual start-up and operation is justified. The generator should 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 system should meet the requirements of Section 4.2.4.6 (4) (iii) (c).

iii. Portable Generating Equipment

Where portable generating equipment or manual transfer is provided, sufficient storage capacity with alarm system should be provided to allow time for detection of pump station failure and transportation and connection of generating equipment. The use of special electrical connections and double throw switches are recommended for connecting portable generating equipment.

4.2.5 Forcemains

4.2.5.1 Velocity and Diameter

At design pumping rates, a cleansing velocity of at least 0.6 m/s should be maintained.

4.2.5.2 Air and Vacuum Relief Valve

An air relief valve should be placed at high points in the forcemain to prevent air locking. Vacuum relief valves may be necessary to relieve negative pressures on forcemains. The forcemain configuration and head conditions should be evaluated as to the need for and placement of vacuum relief valves.

4.2.5.3 Termination

Forcemains should enter the gravity sewer system at a point not more than 600 mm above the flow line of the receiving manhole.

4.2.5.4 Design Pressure

The forcemain and station piping should be designed to withstand water hammer pressures and associated cyclic reversal of stresses that are expected with the cycling of wastewater lift stations. Surge protection systems should be evaluated.

4.2.6 Security of Open Trenches and Excavations

In order to ensure public safety, Local Authorities responsible for the construction should secure open trenches and excavations during non-working periods by installing fences / barricades and / or warning lights / signs.

4.3 Wastewater Treatment

4.3.1 Mechanical Wastewater Treatment

4.3.1.1 Site Selection

1. Plant Location

   Some of the factors which should be taken into consideration when selecting a new plant site are as follows:

   a. setback distances from land use surrounding plant site [see Section 3.4.3.2 (1)];

   b. susceptibility of site to flooding [See Section 3.4.1.3 (3)];

   c. prevailing wind direction; and

   d. adequacy of site for future expansion.

2. Plant Layout

   Plant buildings should be situated to provide adequate allowances for future expansions of the various treatment sections. The plant should also be oriented so that the best advantage can be taken of the prevailing wind and weather conditions to minimize odour, noise, misting, freezing problems, energy consumption, and other environmental impacts. The plant layout should also allow for the probability of snow drifting. Entrances, roadways and open tankage should be located so that the effect of snow drifting on operations will be minimized.

   Processing units should be arranged in a logical progression to avoid the necessity for major pipelines or conduits to transmit wastewater, sludges, or chemicals from one module to the next, and also to provide for convenience of operation and ease of flow splitting for proposed and future treatment units.

   Vehicular access should be sufficient to allow for the largest anticipated delivery or disposal, with allowance made to accommodate vehicle turning and forward exit from the plant site.

3. Provision for Expansion

   In addition to the general site considerations, there are a number of allowances needed to provide for economical and practical expansion of the wastewater treatment facilities. Key provisions include:

   a. Design of on-site pumping stations such that their capacity can be increased and / or parallel facilities constructed without the need for major disruption of the plant's operation;

   b. Layout and sizing of channels and plant piping such that additional treatment units can be added or increases in loading rates accommodated. Similarly, the layout of buildings and tankage should accommodate the location of the future stages of expansion;

c. Space provision within buildings to provide for replacement of equipment with larger capacity units. This is particularly important with equipment such as pumps, blowers, boilers, heat exchanges, etc. Adequate working space should be provided around equipment, and provision made for the removal of equipment; and

d. Sizing of inlet and outlet sewers to account for the ultimate plant capacity. Provided that problems will not occur with excessive sedimentation in the sewers, these sewers should be sized for the ultimate condition. With diffused outfalls, satisfactory port velocities can often be obtained by blocking off ports which will not be required until subsequent expansion stages.

4.3.1.2 Plant Hydraulics

1. Wastewater Pumpage

Raw wastewater and any intermediate wastewater pump stations associated with wastewater treatment works should be capable of conveying the peak wastewater flow rates to downstream treatment units. Pumping equipment should also be designed so that downstream treatment units are not subjected to unnecessary surging. This is best achieved by providing variable capacity, or multiple fixed capacity pumps, so that pump discharge rates will closely match the sewage inflow rate. [See also Section 4.2.4.2 (2) (vii)].

2. Channel Flow

Channels should be designed to convey the initial and ultimate range of flows expected. To avoid solids build-up, the following scouring velocities should be developed in normally used channels at least once per day:

Wastewater containing grit	- 0.9 m/s
Wastewater containing floc suspensions	- 0.45 to 0.60 m/s

Where the above scouring velocities cannot be obtained, channels may be aerated to prevent solids deposition.

3. Flow Division

Within wastewater treatment plants, there will invariably be situations where flow splitting is necessary. Unless certain precautions are taken, the flow will not split in the proportions desired over the full flow range, or the flow may split properly, but the organic load will not be divided in the same proportion.

To ensure that the organic load splits in the same proportion as the flows, the suspended solids should be homogeneously dispersed throughout the liquid and the relative momentum of all particles should be approximately equal at the point of diversion. Some turbulence is therefore desirable before each point of diversion. The following methods can be used to produce homogeneity:

- mechanical mixers;
- diffused aeration;
- bottom entrance into splitting box; or
- bar racks or posts in channels.

4. Plant Hydraulic Gradient

The hydraulic gradient of all gravity flow and pumped waste streams within the wastewater treatment plant, including bypass channels, should be prepared to ensure that adequate provision has been made for all head losses. In calculating the hydraulic gradient, changes in head caused by all factors should be considered, including the following:

a. head losses due to channel and pipe wall friction;

b. head losses due to sudden enlargement or sudden contraction in flow cross section;

c. head losses due to sudden changes in direction, such as at bends, elbows, Y-branches and tees;

d. head losses due to sudden changes in slope, or drops;

e. head losses due to obstructions in conduit;

f. head required to allow flow over weirs, through flumes, orifices and other measuring, controlling, or flow division devices;

g. head losses caused by flow through comminutors, bar screens, tankage, filters and other treatment units;

h. head losses caused by air entrainment or air binding;

i. head losses incurred due to flow splitting along the side of a channel;

j. head increases caused by pumping; and

k. head allowances for expansion requirements and / or process changes.

Consequences of excessive or inadequate allowances for head losses through wastewater treatment works should be noted. If pumpage is required, excessive head loss allowances result in energy wastage. If inadequate head loss allowances are made, operation will be difficult and plant expansion more costly.

4.3.1.3 Wastewater Characterization

Detailed wastewater characterization studies should be undertaken whenever existing data is limited or of suspect quality. Industrial discharges to municipal systems can significantly alter the characteristics and treatability of domestic wastewater. Wastewater containing industrial discharges should therefore be thoroughly characterized before selecting and designing biological treatment process units. Ideally, the undiluted industrial wastewater itself should be characterized so that any spikes originating from that source can be accounted for in the design of the process units. Oil and grease, pH, phosphorus and nitrogen levels should also be determined prior to selection and design of the treatment process.

Where it is found that sewage strengths vary significantly over the year due to excessive infiltration / inflow, population variations and / or seasonal changes in industrial or commercial operations, estimates should be made of the expected average, maximum, and minimum BOD and suspended solids concentrations in the sewage for each month of the year. If nitrification is required, short-and-long-term variations in ammonia and total Kjeldahl nitrogen concentrations should also be estimated.

Biological treatment process units are generally designed using total BOD loadings, however, in some cases soluble BOD loadings may be used where recommended by equipment suppliers. In such cases it is generally assumed that soluble BOD represents a certain fraction of the total BOD. Because wastewater characteristics can vary significantly, the actual ratio of soluble to total BOD should be determined whenever soluble BOD is used for design purposes.

Optimum growth of the microorganisms is dependent on the supply of essential nutrients and trace elements. In addition to carbon, the two most critical elements are nitrogen and phosphorus. To encourage the growth of the organism, it is advisable to maintain a BOD:N:P ratio of 100:5:1. Failure to maintain a balanced nutrient level could result in operational problem. If necessary, nutrients may have to be added to the wastewater to provide a balanced level for microbial growth.

While the design of the plant to treat the domestic component of the total BOD may be straightforward, the potential difficulties with biological stabilization of the industrial wastewater flows should be recognized. Therefore, the industrial flow component should be characterized by using the following ratio:

$$\frac{\text{BOD of the industrial flow at 5 days and } 20^0\text{C}}{\text{BOD of the industrial flow at 20 days and } 20^0\text{C}}$$

Many industrial wastes also contain substances that may exert toxic effects on the organisms. Phenol, cyanide, ammonia, sulphide, heavy metals and many organic compounds may completely inhibit the microbial activity if these concentrations exceed the limit which an be tolerated by the micro-organisms. Thus, waste characterization is extremely important in the design of the plant.

4.3.1.4 Preliminary Treatment

Preliminary treatment consists of screening and grit removal.

Screening is provided as the first treatment stage for the protection of plant equipment against blockage, reduced operating efficiency, or physical damage.

Grit removal is required to prevent the undue wear of machinery and unwanted accumulation of solids in channels, settling tanks and digesters.

1. Screening Devices

    a. Coarse Screens

        i. Where Required

            Protection for pumps and other equipment should be provided by trash racks, coarse bar racks, or coarse screens.

        ii. Design and Installation

- *Bar Spacing*

  Clear openings between bars should be no less than 25 mm for manually cleaned screens. Clear openings for mechanically cleaned screens may be greater than 6 mm. Maximum clear openings should be 50 mm.

- *Slope and Velocity*

  Manually cleaned screens should be placed on a slope of 30 to 45 degrees from the horizontal.

  Approach velocities should be no less than 0.5 m/s to prevent settling; and no greater than 1 m/s to prevent forcing material through the openings, during normal variations inflow conditions.

- *Channels*

  Dual channels should be provided and equipped with the necessary gates to isolate flow from any screening unit. Provisions should also be made to facilitate dewatering each unit. The channel preceding and following the screen should be shaped to eliminate standing and settling of solids.

- *Auxiliary Screens*

  Where a single mechanically cleaned screen is used, an auxiliary manually cleaned screen should be provided. Where two or more mechanically cleaned screens are used, the design should provide for taking any unit out of service without sacrificing the capability to handle the peak design flows.

- *Invert*

  The screen channel invert should be 75-150 mm below the invert of the incoming sewer.

- *Flow Distribution*

  Entrance channels should be designed to provide equal and uniform distribution of flow to the screens.

- *Backwater Effect on Flow Metering*

  Flow measurement devices should be selected for reliability and accuracy. The effect of changes in backwater elevation, due to intermittent cleaning of screens, should be considered in locations of flow measurement equipment.

- *Freeze Protection*

  Screening devices and screening storage areas should be protected from freezing.

- *Screenings Removal and Disposal*

  A convenient and adequate means for removing screenings should be provided. Hoisting or lifting equipment may be necessary depending on the depth of pit and amount of screenings or equipment to be lifted.

  Facilities should be provided for handling, storage, and disposal of screenings. Screenings should be disposed of at the sanitary landfill.

  Manually cleaned screening facilities should include an accessible platform from which the operator may rake screenings easily and safely. Suitable drainage facilities should be provided for both the platform and the storage area.

b. Fine Screens

i. General

Fine screens should have openings of approximately 1.5 mm. The amount of material removed by fine screens is dependent on the waste stream being treated and screen opening size.

Fine screens should not be considered equivalent to primary sedimentation but may be used in lieu of primary sedimentation where subsequent treatment units are designed on the basis of anticipated screen performance. Selection of screen capacity should consider flow restriction due to retained solids, frequency of cleaning, and extent of cleaning. Where fine screens are used, additional provision for removal of floatable oils and greases should be considered.

ii. Design

A minimum of two fine screens should be provided, each unit being capable of independent operation. Capacity should be provided to treat peak design flow with one unit out of service.

Fine screens should be preceded by a coarse bar screening device. Fine screens should be protected from freezing and located to facilitate maintenance.

2. Grit Removal Facilities

Grit removal is usually accomplished by grit channels or aerated grit chambers. Vortex-type (paddle or jet induced vortex) is another not so common type of device used for grit removal.

a. Grit Channels

i. Where Required

In advance of pumping or treating units behind screening devices.

ii. Design and Installation

- *Number of Channels*

  At least two (with one out-of-service, there should be enough capacity in the remaining unit to handle the peak design flow). Provision should be made for isolating and dewatering each unit.

- *Velocity*

  Channels should be designed to control velocities during normal variations in flow as close as possible to 0.3 m/s.

- *Channel Length and Width*

  The length should be adequate to settle 0.2 mm particle with a specific gravity of 2.65 plus 50% allowance for inlet and outlet turbulence. Channel width should be greater than 375 mm.

- *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 (adjustable weir plates are recommended as they can be moved to prevent the sedimentation of organic solids following grit cleaning).

b. Aerated Grit Chambers

i. Design and Installation

Aerated grit chambers for the removal of 0.2 mm, or larger, particles with specific gravity of 2.65, may be designed in accordance with the following parameters:

- *Detention Time*

  2 to 5 minutes at peak design flow rate (the longer retention times provide additional benefit in the form of pre-aeration).

- *Air Supply*

  4.5 to 12 L/m.s, via wide band diffusion header positioned lengthwise along one wall of tank.

- *Tank Dimensions*

  Lower limit of above aeration rates generally suitable for chambers up to 3.7 m deep and 4.3 m wide; wider, or deeper chambers require aeration rates in the upper end of the above range; long, narrow aerated grit chambers are generally more efficient than short chambers and produce cleaner grit; length /

width ratio normally is 1.5:1 to 2:1, but up to 5:1 may be used; depth / width ratio 1:1.5 to 1:2.

- *Desired Velocities*

  Surface velocity should be 0.45 to 0.6 m/s.

- *Grit Handling*

  Grit chambers should be provided with mechanical equipment for hoisting or transporting grit to ground level. Impervious, non-slip, working surfaces with adequate drainage should be provided for grit handling areas. Grit transporting facilities should be provided with protection against freezing and loss of material.

- *Grit Washing*

  Depending upon the method of removal and ultimate disposal, the grit may have to be washed after removal by devices of the type discussed in the previous section.

- *Multiple Units*

  Generally not required unless economically justifiable, or where grit removal method requires bypassing of chamber.

4.3.1.5 Primary Treatment

Primary treatment consists of pre-aeration settling (sedimentation) to remove readily settleable solids, floating materials and scum from raw sewage. This is an important process in sewage treatment, as it reduces the suspended solids content and the load on the biological treatment units.

Sedimentation may be accomplished in horizontal or vertical flow tanks. In a horizontal flow tank, the sewage enters at one end and leaves at the other end. In a vertical flow tank, sewage enters at the centre and flows to periphery of the tank. Sludge settles to the tank floor and is removed mechanically into hoppers where subsequent withdrawal occurs.

Accumulation of scum is to be expected in the primary tank; and a scum baffle / skimmer bar is necessary to prevent the scum from discharging with the effluent.

The tank sizing should reflect the degree of solids removal needed and the need to avoid septic conditions during low flow periods. Sizing of the clarifier should be based on both the average design and peak design flow conditions, and the larger area determined should be used.

Maintenance provisions, including access to equipment, lighting, hose, bibs, etc. should also be provided.

Design Criteria

a. Minimum water depth - 2.1 m

b. Depth to length ratios (rectangular tank) - 1/10 to 1/30

c. Surface loading for tanks not receiving return sludge; based on average design flow - < 0.47 L/s/m^2

d. Surface loading for tanks not receiving return sludge; based on peak design flow - 0.71 to 1.42 L/s/m^2

e. Surface loading for tanks receiving return sludge; based on peak design flow - < 0.47 L/s/m^2

f. Weir overflow rate - 1.74 to 5.21 L/m/s

g. For scum removal, a scum baffle extending at least 150 mm below the surface is necessary close to the overflow weir.

4.3.1.6 Secondary Treatment

1. General

The objective of secondary treatment (mechanical) is to achieve the effluent standards as specified in Section 3.1.2, Table 3.1. It may be accomplished in a suspended growth system, a fixed film system or a coupled system. The following sections will discuss the use of two suspended growth systems: continuous-flow activated sludge process, sequencing batch reactors and one fixed film process, rotating biological contactors.

An integral component of the secondary treatment is the secondary clarifier, which will be discussed in Section 4.3.1.8.

2. Suspended Growth Systems

a. Continuous-flow activated sludge process

i. General

The activated sludge process and its various modifications may be used where wastewater is amenable to biological treatment. This process requires close attention and competent operating supervision, including routine laboratory control. These requirements should be considered when proposing this type of treatment.

ii. Pre-treatment

The minimum level of pre-treatment should include grit removal and screening. Primary settling tanks are required unless demonstrated otherwise. If primary tanks are not provided, then the downstream units should be adequately sized.

iii. Types

A number of modifications of the activated sludge process have been developed. The major types of continuous-flow activated sludge processes include: plug flow (conventional), complete mix, extended aeration, contact stabilization and step feed systems.

iv. Sludge Bulking Control

The control of sludge bulking condition should be a main design objective of an activated sludge system. The use of the following process configuration should be considered:

- Plug-flow reactor

- If a complete-mix system is proposed, a selector basin should be constructed upstream of the complete mix basin. Complete-mix activated sludge systems are generally more vulnerable to sludge bulking for treating domestic wastes.

In addition, provision for addition of chlorine or other chemicals to selectively eliminate filamentous bacteria should be included. The chlorine dosage should range from 3 to 15 kg Cl_2 per 1000 kg MLSS/d. The chlorine addition locations should be chosen such that the chlorine is added efficiently and the biomass receives adequate chlorine exposure.

v. Winter Protection

Due design considerations should be given to minimizing heat loss and to protecting against freezing during winter. Possible design approaches to reduce heat loss during winter conditions include use of diffused aeration instead of surface aeration, reduced surface area by using a deeper tank, tank insulation and provision of covers. If severe climatic conditions are expected, heat loss calculations should be included in the design to assess the aeration basin temperatures under low ambient and wastewater temperatures and low organic loading conditions.

vi. Aeration Basins

- *Sizing*

  The size of the aeration basins should be determined based on the design sludge age using the maximum monthly average BOD loading in the design year. Table 4.13 shows the generally accepted range of sludge age and other parameters such as F/M ratio, mixed liquor suspended solids, aeration tank detention time, aerator loading and sludge recycle ratio for design of the various modifications of the activated sludge process. Consideration should be given to the low loading conditions in the initial operating period.

- *Number of Units*

  Multiple tanks capable of independent operation should be provided for all plants.

- *Dimensions*

  The dimensions of each independent mixed-aeration tank or return sludge re-aeration tank should be chosen so as to maintain effective mixing and utilization of air when diffused air is used.

Liquid depths should not be less than 3 m, except in special design cases.

- *Controls*

  Inlets and outlets for each aeration tank unit should be suitably equipped with valves, gates, weirs or other devices to permit controlling the flow to any unit and to maintain reasonably constant liquid levels. The hydraulic capacity of the system shall permit the maximum instantaneous hydraulic load to be carried with any single aeration tank unit out of operation.

- *Conduits*

  Channels and pipes carrying liquids with solids in suspension should be designed to maintain self-cleaning velocities or should be agitated to keep such solids in suspension at all rates of flow within the design limits.

- *Measuring Devices*

  Devices should be installed for measuring and indicating flow rates of influent sewage, return sludge, sludge wasting, dissolved oxygen, and air to each aeration tank.

TABLE 4.2
TYPICAL ACTIVATED SLUDGE DESIGN PARAMETERS

Process Modification	Flow Regime	Food to Micro-organism Ratio (g BOD/d) / (g MLSS)[d]	Sludge Age (days)	Mixed-Liquor Suspended Solids (mg/l)	Detention Time (hr)	Organic Loading (g BOD/d m^3 tank volume)	Activated Sludge Return Ratio[c]
Conventional	Plug	0.2 to 0.4	5 to 15	1500 to 3000	4 to 8	350 to 650	0.25 to 0.5
Complete mix	Complete mix	0.2 to 0.6	5 to 15	2000 to 5000	3 to 5	350 to 1900	0.25 to 1.0
Step aeration	Plug	0.2 to 0.6	5 to 15	2000 to 3500	3 to 5	350 to 1000	0.25 to 0.75
Contact stabilization	Plug or complete mix	0.2 to 0.6	5 to 15	1000 to 4000[a] 4000 to 10 000[b]	0.5 to 1.5[a] 3 to 6[b]	500 to 1200	0.25 to 1.0
Extended aeration	Plug or complete mix	0.05 to 0.15	10 to 30	2000 to 6000	10 to 24	150 to 400	0.75 to 1.0
High rate	Complete mix	0.4 to 1.5	5 to 10	6000 to 8000	1 to 3	1600 to 4000	0.25 to 0.5
High-purity oxygen systems	Complete mix reactors in series	0.2 to 1.2	3 to 10	3000 to 6000	1 to 5	1600 to 4000	0.25 to 0.5

Notes:
a Contact tank
b Stabilization tank
c 1.5 to 2.0 is required for nitrifying facilities
d < 0.2 is required for nitrifying facilities

- *Conduits*

  Channels and pipes carrying liquids with solids in suspension should be designed to maintain self-cleaning velocities or should be agitated to keep such solids in suspension at all rates of flow within the design limits.

- *Measuring Devices*

  Devices should be installed for measuring and indicating flow rates of influent sewage, return sludge, sludge wasting, dissolved oxygen, and air to each aeration tank.

- *Freeboard*

  Aeration tanks should have a freeboard of at least 0.6 m. Greater heights are desirable. Aeration tanks with mechanical aerators require a minimum freeboard of 1 m.

- *Foam Control*

  Foam control devices should be provided for aeration tanks. Suitable spray systems or other appropriate means will be acceptable. If potable water is used, adequate backflow prevention should be provided on the water lines. The spray lines should have provisions to prevent damage by freezing, where appropriate.

- *Drain and Bypass*

  Provision should be made for dewatering each aeration tank for cleaning and maintenance. The dewatering system should be sized to permit removal of the tank contents within 24 hours. If a drain is used, it should be provided with a control valve. The dewatering discharge should be upstream of the activated sludge process. Provision should be made to isolate each aeration tank without disrupting flow to other aeration tanks.

vii. Activated Sludge Return Equipment

The minimum return sludge rate of withdrawal from the secondary clarifier is a function of the concentration of suspended solids in the aeration tank, the settleability of these solids, and the length of time these solids are retained in the secondary clarifier. The rate of sludge return expressed as a ratio of the average design flow should be variable within the limits set forth in Table 4.2. Separate sludge return lines and pumps should be provided for each clarifier; the system should also be equipped with mechanical or electrical variable speed drive to vary the output of the pump. The pump should be designed for 50 to 125% return of sludge.

Provision should be made in the return lines for the addition of chlorine to the return sludge for controlling sludge bulking.

viii. Waste Activated Sludge Equipment

In designing waste activated control facilities, flexibility should be provided so that the excess activated sludge may be wasted from the return activated sludge lines or directly from the aeration tank. While wasting from the return lines gives a more concentrated sludge, wasting directly from the mixed liquor provides a simpler process control.

The waste activated sludge pumps and pipelines should be sized based on the expected maximum sludge production rates and minimum sludge concentrations. For installations where sludge wastage is not continuous, the sizes of the pumps and pipelines should be increased to handle the sludge wastage during the expected wasting period.

ix. Safety

Handrails should be provided around all aeration tanks and clarifiers and conform to the safety provision of the Alberta Occupational Health and Safety Act and Regulations.

The following safety equipment should be provided near aeration tanks and clarifiers:

- safety vests
- lifelines and rings
- safety poles.

Walkways near aeration tanks should have a roughened surface or grating to provide safe footing.

Sufficient lighting should be provided to permit safe working conditions near aeration tanks and clarifiers at night.

b. Sequencing Batch Reactors

i. General

The sequencing batch reactor (SBR) is a fill and draw activated sludge treatment system. It includes a generic system of variable volume activated sludge in which aeration, sedimentation and decant are combined in a single reactor. Consequently, there are no dedicated secondary clarifier or associated return sludge facilities. The SBR technology is suitable for small installations.

ii. Pre-treatment

The minimum level of pre-treatment should include grit removal and screening.

iii. Types

The SBR systems can be classified under two main types: (1) intermittent feed and intermittent discharge (IFID), and (2) continuous feed and intermittent discharge (CFID).

The IFID systems are sometimes referred to the conventional SBR systems. The common characteristics of all IFID systems is that the influent flow to the reactor is discontinued for some portion of each cycle. The IFID reactor treats the influent wastewater through a succession of operating steps, namely fill, react, settle, draw and idle. The liquid volume inside the SBR increases from a set minimum volume to a predetermined maximum volume during the fill period. Mixing and / or aeration may be provided during this fill step. During the react period, flow to the tank is discontinued and aeration and / or mixing are provided, while sufficient time is allowed for the microbial reactions to take place. During the settle period, quiescent conditions are initiated and the biomass is allowed to flocculate and settle prior to removal. During the draw or decant period, the treated and clarified supernatant is removed from the reactor to the minimum volume level. During the idle period, which is normal component in multi-reactor installations, biomass is retained in the reactor but no waste is treated. During this period, excess biomass may be removed from the tank to maintain the desired sludge age.

The CFID reactors receive wastewater during all phases of the treatment cycle. Because it has continuous fill, it has no separate fill and idle periods. The CFID reactors always have a pre-reaction compartment at the influent end terminating in a baffle.

iv. Winter Protection

The winter protection requirements for the SBR systems are higher than the continuous flow activated sludge systems because of the longer total retention times. In addition to the provisions stated for Suspended Growth Systems (subsection 2.i.e), further considerations should be given to the possibility of freezing of equipment and impact of frozen scum on the proposed decanter system.

v. SBR Basins

- *Sizing*

  The size of the SBR basins should be determined based on the design aerobic sludge age or aerobic food to microorganism (F/M) ratio, using the maximum monthly average BOD loading in the design year. The aerobic sludge age (or F/M ratio) is determined based on the total system sludge age (or F/M) adjusted based on the aerate (aerate fill plus react) in the operating cycle. The aerobic sludge age and F/M ratio should fall within the acceptable range stated in Table 4.2. The MLSS levels of an SBR change throughout the operating cycle. The selected MLSS levels in calculating sludge age or F/M ratios should correspond to the levels during the react period.

- *Dimensions*

  A key design consideration with CFID systems is minimization of short-circuiting between influent and effluent. The reactor should be rectangular in shape with length to width ratios of at least 2:1. Baffling should also be provided. The length to width ratio is generally less critical for IFID system but the exact dimensions may be affected by the choice of influent distribution system.

- *Liquid Depths*

  The top liquid depths should not be less than 3 m, except in special design cases. In most practical cases, the top liquid depths should range between 4 to 6 m. The bottom liquid depth should be designed based on the required fill volume to handle peak flow conditions. The bottom liquid depths should be decided based on the expected sludge settleability.

- *Number of Units*

  An IFID system must comprise a minimum of two SBR tanks or a storage tank and an SBR tank to accommodate continuous inflow. One CFID reactor is adequate to handle continuous flow. However, multiple tanks capable of independent operation should be provided for all plants.

- *Controls*

  Inlets and outlets for each aeration tank unit should be suitably equipped with valves, gates, weirs or other devices to permit controlling the flow to any unit and to maintain reasonably constant liquid level. The hydraulic capacity of the system shall permit the maximum instantaneous hydraulic load to be carried with any single aeration tank unit out of operation.

- *Measuring Devices*

  Devices should be installed for measuring and indicating flow rates of influent sewage, sludge wasting, dissolved oxygen, and air to each SBR tank.

- *Freeboard*

  SBR tanks should have a freeboard of at least 0.6 m. Greater heights are desirable. Aeration tanks with mechanical aerators require a minimum freeboard of 1 m.

- *Foam Control*

  Foam control devices should be provided for aeration tanks. Suitable spray systems with provision for chlorine addition or other appropriate means will be acceptable. If potable water is used, adequate backflow prevention should be provided on the water lines. The spray lines should have provisions to prevent damage by freezing, where appropriate.

- *Drain and Bypass*

  Provision should be made for dewatering each SBR tank for cleaning and maintenance. The dewatering system should be sized to permit removal of the tank contents within 24 hours. If a drain is used, it should be provided with a control valve. The dewatering discharge should be upstream of the SBR process. Provision should be made to isolate each SBR tank without disrupting flow to other aeration tanks.

- *Overflow*

  An overflow system should be provided in the SBR basins to handle extreme flow conditions or equipment malfunction conditions.

vi. Decanter

There are various decanter designs of varying sophistication and complexity proposed by the SBR equipment suppliers. In exposed installations where severe climatic conditions, winter protection should be a major consideration in selecting the decanter design.

Among the recent types are: (i) floating decanter, (ii) fixed decanter, and (iii) mechanically actuated surface skimmer. If a fixed decanter is proposed, longer duration of the settle period should be allowed to ensure that the sludge blanket is located low enough to start each decant cycle.

The decanter should have positive control to prevent solids entry into the decanter during aerate and settle period. The common solids excluding decanters include those: (i) incorporating a spring loaded solids excluding valve, (ii) physically removed from the mixed liquor except during decant period, and (iii) mechanically closed when not in use by a hydraulic or electric motor. If positive control is not provided, the effluent from the decanter should be recycled for at least the first several minutes before discharge. The decanter should have positive control against the entry of scum during the settle period; the decanter should also have a scum baffle to prevent scum from exiting with the effluent.

The size of the decanter should be determined based on the fill volume and the decant period. The decant period is generally one half to one hour.

vii. Waste Activated Sludge Equipment

The wasting of excess activated sludge is generally discontinuous for SBR system. The size of the waste activated sludge equipment should be decided based on the expected wastage period. Consideration should be given to the practical number of operating cycles during the working hours of the operator.

viii. Aeration Equipment

When choosing the aeration equipment for a SBR system, consideration should be given to the intermittent aeration conditions and the possibility of diffuser clogging. Because aeration takes place only during part of the operating cycle, the aeration equipment should be sized such that the required oxygen transfer can be provided during the react / fill and react periods. The temporal variation of oxygen requirements should also be considered.

ix. Downstream Facilities

The design of downstream facilities should allow for the intermittent discharge of SBR effluent. Note that the average decant rate is generally higher than the design peak flow rate because of the intermittent discharge. If the decanting devices are of varying flow rate design, the peak flow rate in the beginning of the decant cycle should be used to determine the hydraulic capacity of the downstream facilities. The impact of the intermittent effluent discharge on the downstream facilities such as UV disinfection should be taken into consideration. An effluent equalization basin should be provided, when appropriate.

x. Safety

Handrails should be provided around all aeration tanks and clarifiers and conform to the safety provision of the Alberta Occupational Health and Safety Act and Regulations.

The following safety equipment should be provided near aeration tanks and clarifiers:

- safety vests
- lifelines and rings
- safety poles.

Walkways near aeration tanks should have a roughened surface or grating to provide safe footing.

Sufficient lighting should be provided to permit safe working conditions near aeration tanks and clarifiers at night.

3. Fixed Film Systems

a. Rotating Biological Contactor

i. General

The rotating biological contactor (RBC) process may be used where wastewater is amenable to biological treatment.

ii. Pre-treatment

Primary clarifiers should be provided ahead of the RBC process to minimize solids settling in the RBC tanks. If the influent contains appreciable amount of sulphide greater than 0.5 mg/L), pre-aeration should be provided upstream of the RBC process.

iii. Media Types

The media used for RBCs are manufactured of high-density polyethylene and are provided in different configurations or corrugated patterns.

The types of media are classified based on the area of media on the shaft and are commonly termed as standard density, medium-density and high-density. Standard density media have surface area of 9300 m^2 per 8.23 m shaft and should normally be used in the lead stages of an RBC process train. Medium and high-density media have surface areas of 11 150 to 16 700 m^2 per shaft and should be used only after the second shaft.

iv. Staging

RBC plants should be designed in multiple stages with sufficient operational flexibility to split incoming flows between stages during peak loading periods so as not to exceed loading limitations to the first stage. A minimum of two stages is required.

v. Design Loading

The typical design loadings for non-nitrifying rotating biological contactors are shown in Table 4.3.

TABLE 4.3
TYPICAL RBC DESIGN LOADING

Organic loading kg CBOD/d/10^3 m^2 of disc surface kg TBOD/d/10^3 m^2 of disc surface	4 to 10 10 to 17
Maximum organic loading in the first stage kg CBOD/d/10^3 m^2 of disc surface kg TBOD/d/10^3 m^2 of disc surface	20 to 30 40 to 60
Hydraulic Loading, m^3/d/m^2 of disc surface	0.02 to 0.08

vi. Enclosures

Enclosures should be provided for the RBC media to prevent algal growth on the media and minimize the effect of cold weather. Enclosures may be either fabricated individual enclosures or building enclosing several shafts. A building, enclosing the units, is preferable to individual enclosures, due to problems of any repair of the individual enclosures in the winter.

Individual enclosures, if proposed, should be made of material resistant to damage from humidity and corrosion. The exterior of enclosures should be resistant to deterioration from direct sunlight. Access points should be provided at each end of the enclosure to permit inspection of shafts and to perform operation and maintenance.

Enclosures should be removable to allow removal of the shaft assemblies. Access around enclosures should be sufficient to permit suitable lifting equipment access to lift covers and shafts.

Buildings should be designed with provision to remove shafts without damage to the structure. Buildings should also be designed with adequate ventilation and humidity control to ensure adequate oxygen is available for the RBC shafts, provide a safe environment for operating staff to perform normal operation and maintenance and minimize the damage to the structure and equipment from excess moisture. Building material and components should be resistant to corrosion.

vii. Hydraulics

The RBC design should incorporate sufficient hydraulic controls, such as weirs, to ensure that the flow is distributed evenly to parallel units. RBC tank design should provide a means for distributing the influent flow evenly across each RBC shaft. Intermediate baffles placed between treatment stages in the RBC system should be designed to minimize solids deposition. The RBC units should be designed with flexibility for series and parallel operation.

viii. Dewatering

The design should provide for dewatering of RBC tanks.

ix. Shaft Drives

The electric motor and gear reducer should be located to prevent contact with the wastewater at peak flow rates. Variable speed drives should be provided.

x. Recycle

Effluent recycle after clarification should be provided for small installations where minimum diurnal flows may be very small. Recycle should be considered in any size plant where minimum flows are less than 30 percent of the average daily design flow.

xi. Load Cells

Load cells should be provided for each shaft. A clean water wet load should be derived at startup to provide for biomass measurement after growth occurs.

4. Aeration

a. Oxygen Requirements

Secondary biological systems should generally be designed to supply oxygen to satisfy the carbonaceous biochemical oxygen demand. However, depending on the local conditions, the designer should also take into account the nitrogenous biochemical oxygen demand and inorganic chemical oxygen demand in calculating the oxygen requirements.

It is likely that nitrification (oxidation of ammonia nitrogen to nitrate nitrogen) will occur during the summer when temperatures are higher; without adequate oxygen, the onset of nitrification can lead to septic conditions and process upsets. Though nitrification is controlled during the summer period by reducing the inventory of MLSS in the aeration tank, some nitrogenous oxygen demand is inevitable and the designer is well advised to provide an allowance for oxygen transfer capability to satisfy these periods of partial nitrification.

Based on the wastewater characteristics, inorganic chemical oxygen demand should also be considered in calculating the oxygen requirements. For instance, hydrogen sulphide will have a demand on oxygen under septic conditions. The designer should evaluate the effect of wastewater septicity and include an allowance for oxygen requirements associated with hydrogen sulphide in the influent wastewater.

i. Carbonaceous Biochemical Oxygen Demand

The carbonaceous oxygen demand may be estimated based on the process oxygen balance using the following formula.

$$R_c = Q\,(S_o\text{-}S)(1 + b\,O_c - BY)/(1+bO_c)$$

where:

R_c = mass oxygen required per unit time to satisfy the carbonaceous biochemical oxygen demand.
Q = flow rate
S_o, S = total carbonaceous oxygen demand of the influent and effluent. The effluent CBOD should be assumed to be zero in the sizing of aeration equipment.
b = endogenous decay coefficient
Y = true cell yield
O_c = solids retention time
B = oxygen equivalent of cell mass

As an alternative, the carbonaceous oxygen requirements based on different sludge age are suggested below:

SRT	Oxygen Required (kg O_2 / kg CBOD)
5	1.0
10	1.1
15	1.2
20+	1.3

ii. Nitrogenous Biochemical Oxygen Demand

The nitrogenous oxygen demand may be estimated by the following equation:

$$R_n = 4.6 \times Q \times (N_o - N)$$

where N_o and N are influent and effluent oxidizable nitrogen respectively. The effluent nitrogen level should be assumed to be zero in the sizing of the aeration equipment.

iii. Inorganic Chemical Oxygen Demand

This oxygen demand is most often estimated based on a stoichiometric calculation.

iv. Spatial and temporal variations

In addition to the total oxygen demand caused by the above sources, the spatial and temporal variations in the demands within the reactor should also be considered in sizing the aeration equipment.

v. Mixing requirement

In addition to oxygen transfer, sufficient mixing should be maintained such that the biological solids are kept in close contact with the wastewater as shown.

TABLE 4.4 MIXING REQUIREMENTS

Type of Equipment	Minimum Mixing Energy
Fine bubble diffused aeration	40 $m^3/min/10^3.m^3$ floor coverage
Coarse bubble diffused aeration	20 $m^3/min/10^3.m^2$ floor coverage
Mechanical	20 $kW/10^3.m^3$

b. Aeration System Alternatives

The aeration equipment can be classified under two categories. The first category is the diffused aeration system. This system supplies oxygen by introducing air into the wastewater with submerged diffusers or other aeration devices. The equipment under this category can be further divided into three groups: porous diffuser system, nonporous diffuser system and other aeration devices such as aspirators or jet aerators.

The second category is the mechanical aeration system, which supplies oxygen by agitating the wastewater mechanically so as to promote solution of air from the atmosphere. Mechanical aerators are usually divided into two major groups: aerators with a vertical axis and aerators with a horizontal axis. Both groups are further subdivided into surface and submerged aerators.

The selection of the aeration equipment should be based on both cost and non-monetary considerations. Under cold climatic conditions, special care should be given to freeze protection if surface aerators are proposed.

i. Diffused Air Systems

Air volume requirements for channel, pumps, or other air-use demands should be added to the oxygen requirements of the activated sludge process.

The specified capacity of blowers, particularly centrifugal blowers, should take into account that the air intake temperature might reach extremes and that pressure might be less than normal. Motor horsepower should be sufficient to handle the minimum and maximum ambient temperature. Piping head loss must also be accounted for.

The blowers should be provided in multiple units, arranged and in capacities to meet the maximum air demand with the largest unit out of service. The design should also provide for varying the volume of air delivered in proportion to the load demand of the plant.

The diffusers should be spaced in accordance with the oxygen and mixing requirements in the basin and should be designed to facilitate adjustments of their spacing without major revision to air header piping. The arrangement of the diffusers should also permit their removal for inspection, maintenance, and replacement without dewatering the tank and without shutting off the air supply to other diffusers in the tank. Slip fittings should not be used. Pipe vibrations should be dampened.

Individual units of diffusers should be equipped with control valves, preferably with indicator markings for throttling and complete shutoff. Diffusers in each assembly should have substantially uniform pressure loss.

Flow meters and throttling valves should be placed in each header. Air filters should be provided to prevent clogging of the diffuser system.

For further details, the designer should refer to the USEPA publication entitled Fine Pore Diffused Aeration Manual.

ii. Mechanical Aeration Systems

The mechanism and drive unit should be designed for the expected conditions in the aeration tank in terms of the proven performance of the equipment.

Due to high heat loss, consideration should be given to protecting subsequent treatment units from freezing where it is deemed necessary. Multiple mechanical aeration unit installations should be designed to meet the maximum oxygen demand with the largest unit out of service. The design should normally also provide for varying the amount of oxygen transferred in proportion to the load demand on the plant.

A spare aeration mechanism should be furnished for single-unit installations.

c. Flexibility and Energy Conservation

The design of aeration systems should provide adequate flexibility to vary the oxygen transfer capability and power consumption in relation to oxygen demands. Particular attention should be given to initial operation when oxygen demands may be significantly less than the design oxygen demand. The design should always maintain the minimum mixing levels; mixing may control power requirements at low oxygen demands.

Dissolved oxygen probes and recording should be considered for all activated sludge designs. For larger plants, consideration should be given to automatic control of aeration system oxygen transfer, based on aeration basin dissolved oxygen concentration.

Watt-hour meters should be provided for all aeration system drives to record power usage.

Energy conservation measures should be considered in design of aeration systems. For diffused aeration systems, the following should be considered:

- use of smaller compressors and more units
- variable-speed drives on positive-displacement compressors
- intake throttling on centrifugal compressors
- use of timers (minimum mixing should be maintained)
- use of high-efficiency diffusers

For mechanical aeration systems, the following should be considered:

- use of smaller aerators
- variable aeration tank weirs
- multiple-speed motors
- use of timers.

4.3.1.7 Tertiary Treatment

1. General

   The objective of the tertiary treatment (mechanical) is to achieve the effluent standards as specified in Section 3.1.2, Table 3.2. The tertiary treatment entails nutrient (phosphorus and ammonia) control and effluent disinfection in addition to the reduction of carbonaceous biochemical oxygen demand and total suspended solids. The selection of the most appropriate process is dependent on the following factors:

   - existing process, if any
   - wastewater characteristics
   - space and hydraulic constraints
   - operator's preference.

2. Phosphorus control

   Phosphorus control can be achieved biologically or chemically. The selection of the most appropriate control method should be made based monetary and process considerations. The important factors which impact on the costs comparison include: (1) influent phosphorus levels and loadings, (2) chemical costs, (3) sludge disposal costs, and (4) amenability of Biological Nutrient Removal (BNR) retrofit of the existing process. In addition to the monetary comparison, there are process advantages associated with the BNR process which should be considered:

   - improved sludge settleability
   - recovery of alkalinity (for denitrification).

   a. Biological Phosphorus Removal (BPR)

      Biological Phosphorus Removal is accomplished by a group of organisms that have the ability to uptake quantities of phosphorus in excess of their synthesis requirements when stressed by environmental conditions. This is termed "luxury uptake" and occurs when anaerobic conditions are present in the influent region of a plug flow reactor and aerobic conditions elsewhere. Under these conditions, the phosphorus content of the mixed liquor increases from approximately 1.5 percent to as high as seven (7) percent solids. Phosphorus is removed via wasting sludge. The luxury uptake phenomenon is actually part of an energy storage cycle that allows the phosphorus accumulating organisms to become active in the anaerobic zone.

There are several factors that can affect the phosphorus removal efficiency of the BPR systems. These factors relate to wastewater characteristics, system design and operational methods. These factors can be divided into the following: (i) environmental factors, such as temperature, D.O. and pH, (ii) substrate availability as affected by influent wastewater characteristics, the level of volatile fatty acid (VFA) production, and the presence of nitrates, (iii) design parameters, such as system Sludge Retention Time (SRT), anaerobic zone detention time, and aerobic zone detention time, and (iv) effluent total suspended solids concentration.

There are various process configurations to achieve BPR. The process configurations include A/O, A2/O, Modified Bardenpho, UCT, VIP, Step Bio-P, PhoStrip and Bio-denipho and Trio-denipho processes. Some processes are proprietary. Most processes require different degrees of plant capacity derating to achieve BPR, when retrofitting existing wastewater treatment plants.

It is not the intention of this section to provide step-by-step guidelines for the design of a BNR system. However, adequate design consideration should be given to the following:

- account for the effect of sidestream returns
- avoid trapping of Nocardia / foam in the bioreactors
- provide flexibility of zonal hydraulic retention times (HRTs)
- provide good D.O. control of all aerobic cells
- include by-pass capability to reduce unnecessary tankage during initial or low load conditions
- provide flexibility of recycle sources and destinations
- pay careful attention to mixing energy and mixer placement
- avoid secondary release of phosphorus (release without energy uptake).

For additional details, the designer should refer to the USEPA publication entitled <u>Design Manual on Nitrogen Control and Phosphorus Removal</u>.

b. Chemical Phosphorus removal

Phosphorus in wastewater can be removed chemically by the addition of metallic salts or lime. Lime addition is seldom practiced today due to the high chemical usage, problems associated with handling lime and the large volume of sludge generated from lime addition.

The metallic salts commonly used for phosphorus removal are aluminum or iron salts. Iron salts are less common for use in Alberta because of their costs, and limited availability. The common aluminum salts for phosphorus removal include alum, sodium aluminate, polyaluminum chloride and Polyaluminum Silicate Sulphate. Generally, the best removal result is achieved when an organic polymer is added with the metallic salts to assist the precipitation process. Jar tests should be carried out to determine the most appropriate chemicals or their combination and the optimal dosage.

The metallic salts and polymer can be added before the primary clarifier, before or inside the aeration basin and before the final clarifiers. It is a good practice to provide multiple feed points to improve process flexibility. The locations of the feed points should be chosen such that the flow conditions are turbulent to promote good mixing of chemicals.

3. Ammonia Removal

Ammonia can be removed either biologically or chemically using air stripping or breakpoint chlorination. Chemical method for ammonia removal is generally less cost effective and should be considered only under special cases.

The biological removal of ammonia is achieved by the biochemical oxidation of ammonia to nitrate with nitrite as an intermediate. Two autotrophic microorganisms, Nitrosomonas and Nitrobacter are responsible for these reactions. These bacteria grow slower than the BOD removal bacteria and are sensitive to low temperature and adverse environmental conditions. The most important requirement for achieving consistent nitrification is to maintain adequate sludge retention times to prevent these slow growing bacteria from washing out from the biological systems.

Alkalinity is consumed in the nitrification process (7.4 mg of alkalinity as $CaCO_3$ for every mg of NH_4-N nitrified). This loss of alkalinity may have an impact on the design of nitrifying plants treating poorly buffered or low alkalinity wastewater, because the nitrifiers also tend to be pH dependent.

Nitrification can be achieved by either a suspended growth or fixed film process. By incorporating dentrification into the design, approximately two thirds of the oxygen required for nitrification can be recovered.

4. Effluent Disinfection

a. General

i. Methods of Disinfection

Bactericides, Viricides and Potential Disinfectants

UV radiation, chlorine and bromine chloride may be used in full-scale plants to reduce microorganisms (bacteria and viruses) in the wastewater effluent.

Disinfectants

Ozone and chlorine dioxide may be used in full-scale plants to reduce microorganisms (cysts, bacteria, and viruses) in the wastewater effluent.

ii. Selection Method

The selection of a disinfection method should be based on both monetary and non-monetary considerations. In conducting cost comparison, capital, O&M and total life-cycle costs should be estimated. The non-monetary considerations should include disinfection effectiveness, state of development, effluent quality impacts, chemical hazards and safety concerns, process complexity and ease of operation and maintenance.

b. UV Radiation

i. General

UV radiation may be used to achieve the disinfection requirements to produce effluent that meets Alberta surface water bacteriological quality for recreational waters. At the UV dose commonly applied, this process is effective in inactivating indicator bacteria such as those members of the coliform group, and to a lesser extent the viruses. At this dose, UV is not effective in inactivating protozoa.

ii. Types of UV Systems

UV disinfection systems can be broadly classified under three groups based on the types of lamps used: (A) low-pressure, low-intensity, (B) low-pressure, high-intensity, and (C) medium-pressure, high-intensity UV systems. The sources of UV radiation of these UV systems are mercury vapour lamps, which are operated at different mercury vapour pressures and discharge currents.

Low-pressure, high-intensity systems are uncommon and not widely used.

A. **Low-Pressure, Low-Intensity Systems**

Of the disinfection systems using low-pressure, low-intensity mercury lamps, there are three major different reactor designs: open channel, closed chamber and Teflon tube. In the open channel systems, the lamps can be oriented either horizontal and parallel-to-flow or vertical. Other reactor configurations such as closed chamber or Teflon tube should be considered under special cases only.

B. **Medium Pressure, High Intensity Systems**

There are three designs using this type of lamps: horizontal and parallel-to-flow, horizontal and perpendicular-to-flow or closed chamber.

iii. Selection of UV Disinfection Alternatives

The factors which affect the selection of the appropriate UV system include:

- design flow rate
- wastewater transmittance and suspended solids levels
- effluent bacteria standards

- continuous or seasonal disinfection
- lamp fouling potential
- land and hydraulic constraints
- power charges
- labour costs
- headloss constraints.

Generally, a low-pressure, low-intensity system is suitable for small installations (less than 300 lamps). For major installations, detailed cost comparison of the promising alternatives should be conducted before the UV system is selected.

iv. Influent Characterization

The design of a UV disinfection facility requires a thorough understanding of the characteristics of the influent to the UV facility. The important parameters include flow rates, UV transmittance, total suspended solids and coliform levels. These parameters have direct impact on the UV disinfection efficiency. Other parameters which should also be considered include BOD, ammonia, nitrate, iron, hardness, oil and grease and wastewater temperature. They either have indirect impact on disinfection efficiency or direct impact on lamp fouling.

When a UV disinfection facility is added to an existing treatment plant, the historical data regarding effluent qualities should be studied. Statistical and probabilistic analyses should be conducted to establish the expected worst-case operating conditions as the design basis. Measurement of UV transmittance levels should also be conducted.

If UV disinfection is proposed for a new plant, the effluent qualities of similar installations and available published information may be used as the design basis; or pilot plant studies should be considered. The presence and nature of any industrial discharge and the raw water source and quality should be considered. The effluent UV transmittance depends on the type and degree of treatment. Generally, a higher degree of treatment gives a higher light transmittance of the effluent. The UV transmittance from a suspended growth process is generally higher than that from a fixed film process. Typical ranges of UV transmittance values for various secondary processes are shown in Table 4.5.

TABLE 4.5
RANGE OF UNFILTERED UV TRANSMITTANCE

Treatment Processes	Typical Range
Conventional activated sludge	35 to 65 percent
Fixed film process	30 to 50 percent
Lagoon	30 to 50 percent
Secondary plus filtration	50 to 70 percent

v. Assessment of Minimum UV Dose Requirements

UV dose is defined as the product of the intensity of radiation (microwatts per square centimeter) and the length of time (seconds) during which the wastewater is exposed to UV radiation. The minimum UV dose requirement is dependent on the design influent characteristics and the effluent coliform standards.

The minimum UV dose requirements may be determined using the following methods:

- *Pilot Testing*

  Pilot testing should be conducted if (A) the wastewater treatment plant (WWTP) is treating significant industrial wastes, (B) it is designed for major installations, or (C) a less common UV system is considered. The pilot-testing programme should include an intensive test programme to develop the dose response relationship of the UV system and a routine test program to assess the lamp sleeve fouling potential.

- *Disinfectability Studies*

  The disinfectability of the wastewater may be assessed in a laboratory using a collimated beam test apparatus. The results may be used to develop the dose response relationship under ideal laboratory conditions. Considerations should be given to allow for the less perfect hydraulic design of the full-scale UV reactor.

- *Mathematical Models*

  The minimum dose requirements may be determined using mathematical models such as those developed by USEPA or WERF. When properly calibrated using the data collected from the pilot studies, these models will provide a useful means for estimating the required dose under the design influent conditions. If the models are not calibrated using the actual wastewater characteristics, adequate safety margin should be allowed in the design.

- *Design and Operating Data from Similar Installations*

  The design and historical operating data of some existing UV installations may be used as the design basis. The data to be used in analyses are the influent and effluent coliform data, TSS levels, UV transmittance and flow rates. Lamp cleaning frequency records should also be reviewed. The important criteria for selecting the similar installations include the degree of treatment, the types of treatment processes and industrial components of the wastewater.

vi. Development of Design Dose and Lamp Requirements

The UV dose used to design the UV facility should be adjusted to account for aging and fouling effects of the UV lamps. The minimum UV dose developed based on clean and new lamps should be increased by the following formula:

Design UV dose = minimum UV dose/Fp/Ft

where F_p = the lamp output reduction factor. This factor is the fraction of initial output the lamp is expected to have at the end of its useful life.

F_t = the quartz sleeve transparency reduction factor. This factor is the fraction of lamp emission that is transmitted through the quartz sleeve immediately before the sleeve is cleaned. It includes the UV output loss due to the clean quartz sleeve and the losses due to the deposits on the sleeve (fouling).

Table 4.6 shows the suggested correction factors for the low-pressure UV systems. The appropriate correction factors for the high intensity UV systems should be decided from the full- or pilot-scale operating data.

TABLE 4.6
DESIGN CORRECTION FACTORS

UV Systems	F_p	F_t (1)
Low-pressure, low-intensity	0.65	0.71

Note:
The power correction due to loss in transmittance is based on clean quartz sleeve loss of 11 percent and the transmittance losses due to sleeve fouling of 20 percent for low-pressure systems.

The lamp requirements of the UV facility should be calculated based on the design UV dose and the design peak flow rate. Standby UV disinfection capacity is generally not required at peak flow conditions to account for units out of service. UV disinfection equipment cleaning or maintenance procedures that require removal of equipment from service should be able to be completed during the expected low flow conditions. Where peak flows are frequent or unpredictable, or the installations only have limited lamps, standby equipment during peak flow conditions should be provided.

vii. Lamp Arrangements

UV lamps are generally grouped in units of modules. The modules are then grouped into lamp banks. Multiple lamp banks are placed in a UV channel and the complete UV facility consists of one or more UV channels.

The choice of UV lamp numbers in a module is generally limited by the design of the UV system. The number of UV lamps in a low-pressure, low-intensity system of horizontal lamp design may range from two to 16 lamps. The number of UV lamps in a low-pressure, low-intensity system of vertical lamp design is typically 40. If a 16 horizontal lamp module or a 40 vertical lamp module is proposed, a mechanical lifting device should be installed to facilitate lamp removal and installation.

The numbers of UV modules in a UV bank are generally determined by hydraulic considerations. The appropriate number of UV modules (and number of lamps per module for horizontal lamp system) should be selected so as to ensure that the UV reactor is conducive to plug flow.

The number of UV banks in a channel should be at least two or four for horizontal and vertical lamp systems, respectively to maintain plug flow characteristics and minimize short-circuiting. For low-pressure, low-intensity systems, two UV channels should be provided to maintain system reliability.

In designing the lamp arrangement, a main important consideration is to facilitate flow pacing. The lamps should be arranged in such a way that the lamp banks can be turned on and off easily to match the expected disinfection needs. Some newer vertical-lamp systems are designed such that individual rows of lamps within a given module may be turned on and off based on flow rate signals.

viii. Reactor Designs

- *Inlet and Outlet*

  UV disinfection reactors should be designed with inlet channel approach and outlet conditions that promote plug flow within the system. To ensure proper inlet and outlet flow conditions, the following criteria are suggested:

  - Unobstructed approach channel length before first UV bank not less than 2 times channel water depth or 1.2 m.
  - Unobstructed downstream channel length following last UV bank before water level control device not less than two (2) times channel water depth or 1.2 m.
  - Spacing between UV banks = minimum spacing required for maintenance and access

- *Flow Distribution*

  If more than one UV channel is provided, a positive flow distribution system such as weirs should be used to ensure equal flow splitting. The influent chamber should be sized such that the headloss along the chamber is less than one tenth of the headloss at each channel under the expected range of flow conditions.

- *Velocity Distribution*

  For low-pressure, low-intensity system, perforated stilling plates should be provided upstream of the first lamp UV lamp banks to ensure uniform velocity distribution.

- *Water Level Control*

  Water depth control devices should be provided in the low pressure, low-intensity systems to maintain a constant water level (plus or minus 2.5 cm) under the expected flow ranges. Weighted adjustable flap gates are generally suitable. If the flow rates during the initial years of operation are significantly lower than the design flow rates, a fixed weir should be installed initially. The water depth control for a medium-pressure, high-intensity system is less critical because its main function is to ensure full submergence of the reactors at all times. Weighted adjustable flap gates and adjustable weir gates are acceptable.

- *Isolation Gates*

  Sluice or weir gates should be provided upstream and downstream of the UV banks to isolate the UV channel when maintenance is needed. Flap gates which may leak under low flow conditions should not be used as isolation gates.

- *Drainage*

  A drainage (mud) valve should be provided in each UV channel for dewatering.

- *Hydraulics*

  Adequate hydraulic head should be allowed for in the design of the UV facility. The main hydraulic head losses occur at the inlet for flow splitting and at the outlet gates for free flow discharge. The losses at the UV banks and reactors are generally small when compared with other losses.

- *Flood Level*

  The top level of the UV process area should be located above the 1:100 year flood level. The channel depths should be checked such that the UV modules can be conveniently lifted up from the main process area.

ix. Lamp Cleaning

Adequate facilities should be provided to facilitate cleaning of lamp sleeves. In addition to the cleaning chemical system, appropriate wash down area and lifting devices should be provided.

For low-pressure, low-intensity systems, lamp cleaning can be performed manually or with a dip tank for small installations. In large installations, a cleaning chemical (acid) bath should be provided so that one lamp bank can be cleaned at a time. If in-channel lamp cleaning system is proposed, suitable concrete lining and isolation gates should be provided. Gates should preferably be of stainless steel or fibreglass.

For medium-pressure, high-intensity systems, cleaning of lamps may be performed by the built-in wiping mechanisms, as included in some designs. If only a mechanical wiper is provided, provision should be made for cleaning the lamps chemically on a regular basis.

x. Screens

Screens may be required immediately upstream of the UV facilities to remove algal clump or other objects which may impact UV disinfection performance, or damage the lamps. Screen openings may range from 2 to 13 mm, depending on the selected types of UV systems, size of installations, the expected degree of algae problem and availability of operating resources. Mechanical screens should be considered for large installations.

xi. Ballasts

The ballasts should be compatible with the proposed lamp type. If possible, the more energy efficient electronic ballast should be specified.

xii. Control and Instrumentation

The choice of the most appropriate control system for a UV facility is dependent on the size of the installation, the available operating staff and the control system for other WWTP facilities. For major installations, the functions of a UV disinfection control system should include the following:

- activate and deactivate UV banks and channels based on the disinfection needs
- activate and deactivate lamp cleaning mechanism (for medium-pressure, high-intensity systems)
- monitor influent characteristics and equipment operation status
- generate alarms
- monitor UV intensity.

The major parameters to be monitored continuously in the control system may include flow rate, UV transmittance, UV intensity, water levels, UV output and lamp / bank status. Other important information for process control such as influent and effluent coliform levels and total suspended solids levels will be collected by taking samples manually.

For small installations, the minimum function of a control system should include flow pacing of the UV lamp banks to optimize the UV dose. While overdosing of UV radiation may not create any harmful environmental by-products, it will increase the O&M costs necessarily by wasting power and reducing useful lamp life.

xiii. Housing

The need for housing of the UV facility should be decided on a case-by-case basis. Under extreme climatic conditions, housing is generally desirable for low-pressure systems where manual lamp cleaning is required.

If housing is provided, attention should be paid to humidity control and ease of equipment transport into and within the building.

xiv. Sequencing Batch Reactor

If the UV disinfection is proposed with a sequencing batch reactor system, consideration should be paid to the intermittent effluent discharge. The design flow rates should be decided based on the decant flow rates. If decant system is not designed to have constant-flow design, the maximum flow rate during the decant cycle should be used to size the UV disinfection. It may be appropriate to provide an upstream equalization basin to provide continuous flow through the UV system.

The number of expected on-off operating cycles of the UV lamp banks should be estimated. If the expected number of on and off is high, provisions should be made of an equalization basin upstream of the UV facilities or recycle pumps downstream of the UV facilities.

xv. Safety

Three safety issues should be addressed fully in the design of a UV disinfection facility. They are: (1) exposure to UV radiation, (2) electrical hazards, and (3) handling of acids.

Exposure to UV radiation may affect the eyes with a temporary painful condition known as conjunctivitis or "welder's flash". Bare skin will also be burned upon exposure to UV at these wavelengths. For the low-pressure, low-intensity UV systems, eye shields should be provided in the UV channels. The use of chequer plate instead of open floor grating should also be considered. The medium-pressure, high-intensity systems must be completely enclosed. All UV systems must be equipped with safety interlocks that shut off operating modules if they are removed from the channel.

The safety requirements for handling high-voltage electricity should be followed. All the electrical components should be designed for submergence or located above the 1:100 flood levels. Electrical hazards should be minimized by the inclusion of ground fault interruption circuitry with each operating module.

Due considerations should be given to the cleaning chemical tank design such that the chance of accidental falling into the tank is minimized. Handrails should be provided when needed. Safety shower and eye wash facilities should be provided in the chemical bath and chemical handling areas.

c. Ozonation

i. General

Ozonation is generally applied only to effluents that are nitrified, highly clarified (filtered) or both. This method may also be considered for unfiltered secondary effluents when a low-cost source of oxygen is available, e.g., at a pure oxygen activated sludge WWTP.

ii. Source of Ozone

Ozone must be generated on site because it is chemically unstable and decomposes rapidly to oxygen after generation. The most efficient method of producing ozone is by electrical discharge using either air or pure oxygen.

iii. Design Requirements

The design requirements for ozonation systems should be decided based on pilot testing or similar full-scale installations. As a minimum, the following design factors should be considered:

- ozone dosage
- dispersion and mixing of ozone in wastewater
- contactor design
- control of off-gas.

iv. Gas Selection and Preparation

Ozone may be generated from air, oxygen-enriched air or high purity oxygen. The concentration of ozone produced increases 2 to 2.5 times when air is replaced by high-purity oxygen at the same gas flow rate. The selection of feed gas should be decided based on cost and other considerations. Generally, the use of pure oxygen is not economical unless high-purity oxygen is required elsewhere in the treatment facility.

Regardless of whether air or high purity oxygen is used as the feed gas, gas preparation is required to ensure it is free of oil, dust and moisture. When ambient air is used as the feed air, the following processing and control units should be provided prior to ozone generation:

- filter - 5 μm modular or fabric filter
- pressurizer - blower, compressor
- after cooler
- oil coalescer - if an oil-free pressurization device is not used
- refrigerant drier - to reduce the size of desiccant drier

- desiccant drier - silica gel, activated alumina, or crystalline zeolite
- filter - 99 percent efficient at 1.0 μm size
- hygrometer
- gas flow meter
- pressure release valve - in high-pressure systems.

When oxygen enriched air is used, the gas preparation units are similar to those required for ambient air except that the desiccant drier is not needed and replaced by a pressure swing separator.

In the case of high-purity oxygen, the following units should be provided prior to ozone generation:

- high-purity oxygen source -cryogenic plant, pressure swing absorption unit or oxygen cylinders
- pressurizer - blower, compressor
- after cooler
- oil coalescer - if an oil-free pressurization device is not used
- refrigerant drier - to reduce the size of desiccant drier
- desiccant drier for recycled oxygen and
- filter - 99 percent efficient at 1.0 μm size
- pressure release valve - in high-pressure systems.

v. Ozone Generation

- *Types*

  The ozone generators may be broadly classified based on their power supply, i.e., low frequency (60 Hz), medium frequency (up to 600 Hz) and high frequency. These types may be further divided based on the cooling media (air, water or both) and the physical arrangements of the dielectrics.

  Small systems (less than 450 kg/d) generally call for the application of low frequency water-cooled units. In larger systems, especially those incorporating high purity oxygen process, the added cost and complexity of higher frequency generators with associated chilled water-cooling systems may be justified.

- *Sizing*

  The total capacity of the ozone generators should be decided such that the required applied ozone dosage can be delivered under peak flow condition. The applied ozone dosage should be calculated based on the required absorbed ozone dosage and the minimum ozone transfer efficiency at the expected ozone dosage and wastewater quality conditions.

  Standby ozone generator capacity is generally not required at peak conditions. Ozone equipment maintenance that requires removal of equipment from service should be able to be completed during expected low flow or dose requirement conditions. Where peak flows are frequent or unpredictable, stand-by equipment during peak flow condition should be considered.

  The number and size of generator units should be decided such that the system can satisfy both the maximum and minimum ozone production rates under the expected operation conditions.

vi. Ozone Contacting

The ozone-contacting basin should be designed such that there is an efficient mass transfer of ozone out of gas bubbles into the bulk liquid and sufficient time for disinfection. Common contactor types include:

- diffused bubble (concurrent and counter current)
- positive pressure injection
- negative pressure (venturi)
- mechanically agitated
- packed tower.

For diffused bubble systems, the contactor should be at least 6 m deep for secondary effluent at an applied ozone dosage of less than 6 mg/L and an elevation of approximately 1000 m in Alberta. The contactor may be deeper if the wastewater to be disinfected is of higher quality, if the applied ozone dosage is higher or if the plant is located at a higher elevation.

Multiple staged ozone contactors should be provided to minimize the effect of short-circuiting. A minimum of three and preferably more stages should be provided. Each stage should be positively isolated from the other to simulate plug flow characteristics and minimize the potential for short-circuiting. The minimum contact time should be six minutes and preferably 10 minutes at design flow rates.

The off-gases from the contact chamber must be treated to destroy any remaining ozone. The product formed by destruction of the remaining ozone is pure oxygen, which may be recycled if pure oxygen is being used to generate the ozone.

vii. Housing

If a building is not provided separately for ozone generation equipment, a gas-tight room should be constructed to separate the ozone equipment from other part of the building. Doors to the ozone generation room should open only from the outside of the building and should be equipped with panic hardware.

At least two means of exit should be provided from each separate room. All exit doors should open outward.

A clear glass, gas tight window should be installed in an exterior door or interior wall of the ozone generation room to permit the ozone generator to be viewed without entering the room.

viii. Ventilation

For ozonation system rooms, continuous mechanical ventilation should be provided to maintain at least six air changes per hour. The entrance to the air exhaust duct from the room should be near the floor and the point of discharge shall be selected such that it will not contaminate the air inlet of any buildings or inhabited areas.

ix. Corrosion Protection

The selection of material should be made with due consideration for ozone's corrosive nature. Only materials at least as corrosion-resistant to ozone as grade 304 L or 316 L stainless steel should be specified for piping containing ozone in non-submerged applications. Unplasterized PVC may be used in submerged piping, provided the gas temperature is below 60°C and the gas pressure is low.

Piping systems should be as simple as possible, specifically selected and manufactured to be suitable for ozone service, with a minimum number of joints. Piping should be well supported and protected against temperature extremes.

x. Safety

The safety issues that should be addressed fully in the design of ozonation system include: (1) exposure to ozone, (2) noise, and (3) electrical hazards.

The occupational exposure of ozone should be controlled such that workers will not be exposed to ozone concentrations in excess of 0.2 mg/m^3 for eight (8) hours or more per workday, and that no worker be exposed to a ceiling concentration of ozone in excess of 0.6 mg/m^3 for more than 10 minutes.

All ozone systems should be provided with an ambient ozone monitor or monitors which are set up to measure the ozone concentration at potential ozone-contaminated locations within the plant. The monitors should be set up to sound an alarm when the ozone concentration reaches 0.2 mg/m^3 and should be set up to shut down the ozone system when the concentrations exceed 0.6 mg/m^3.

Eyewash basins should be provided to enable the operator to rinse ozone from the eyes, if needed. The basins should be located in the outside of the ozone generator room.

Some ozone generation systems are classified as noisy installations. Generally, the main source of noise is the feed-gas compressor. Wherever practical, the feed-gas compressors should be isolated in a sound insulated room.

The ozone generators generally require high power consumption. All safety requirements regarding handling of high voltage power should be followed.

d. Chlorine

i. General

Chlorine is an effective disinfectant. Potential drawbacks to its use include:

- toxicity to aquatic, estuarine and marine organisms
- generation of harmful chlorinated disinfection by-products
- safety concerns during transportation, storage and handling, particularly with gaseous chlorine.

  If chlorination is proposed, due consideration should be given to the above factors. A dechlorination system should be provided.

ii. Forms of Chlorine

Chlorine may be added to the wastewater in the form of liquid / gaseous chlorine or sodium hypochlorite.

iii. Design Requirements

In sizing a chlorination system, the following factors should be considered:

- contact time
- concentration and type of chlorine residual
- mixing
- pH

- suspended solids levels
- temperature
- coliform levels
- ammonia concentrations

- Dechlorination system.

The design should provide adequate flexibility in the chlorination and control system to allow controlled chlorination at the expected flow ranges in the design period. Special consideration should be given to the chlorination requirements during the initial years of operation to ensure the chlorination system is operable at less than design flows without over-chlorination.

iv. Chlorine Addition

Chlorine should be added into the wastewater where good mixing is achieved at all times. Mixing may be accomplished mechanically or hydraulically.

When mechanical mixing is proposed, the following criteria apply:

- a mixer-react unit is necessary that provides 0.1 to 0.3 minutes contact

- chlorine should be injected just upstream of the mixer with a diffuser

- the minimum mixer speed should be 50 revolutions per minute

- the diffuser should be set at least two feet below the minimum wastewater level at low flows

- turbulent flow after complete chlorine mixing should be avoided to prevent chlorine stripping.

Hydraulic mixing should be achieved based on the following criteria:

- Pipe Flow

    - A Reynolds number of greater than or equal to 1.9×10^4 should be achieved at all flow rates. Hydraulic jumps or baffles may be used to create turbulence.

    - A diffuser, with orifice velocities of 5 m/s at peak flows should be provided.

    - The diffuser should be set as deep as possible and at least two feet below minimum wastewater level at low flows.

- Open Channel

    - A hydraulic jump with a minimum Froude number of 4.5 is necessary to provide the adequate hydraulic mixing. Multiple points of chlorine injection should be provided because the jump location may change with changes in flow rates. A parshall flume is not a satisfactory location for hydraulic mixing.

v. Contact Basin

Contact chambers should be sized to provide sufficient retention time for the effluent to meet the required bacteriological quality.

The contact chambers should be baffled to minimize short-circuiting and back mixing. Baffles should be constructed parallel to the longitudinal axis of the chamber with a minimum length-to-width ratio of 40:1. Side water depths should range between 2 to 5 m.

vi. Dechlorination

Dechlorination may be achieved by the use of detention ponds or by the addition of sulphur dioxide or sodium metabisulphite to the chlorinated effluent.

The required sulphur dioxide dosage for dechlorination is 1 mg/L SO_2 for 1 mg/L chlorine residual. Reaction time is essentially instantaneous. Detention time requirements are decided based on the time necessary to ensure complete mixing of the sulphur dioxide. To ensure continuous compliance of the maximum chlorine residual requirements, over-dechlorination followed by re-aeration should be considered. Continuous monitoring of the effluent would be a requirement.

vii. Chlorination equipment and chlorine room design requirements

For details of chlorine equipment requirements and chlorine room design requirements, see Section 1.5.3.1 (2) and (3).

4.3.1.8 Secondary Clarifier

Design of a secondary clarifier for suspended growth systems is different from the design of secondary clarifier for fixed growth systems, in that, to perform properly while producing a concentrated return flow, a suspended growth secondary clarifier should be designed to meet thickening as well as solids separation requirements. Since the rate of recirculation of return sludge from the final settling tanks to the aeration is quite high in activated sludge processes, surface overflow rate and weir overflow rate should be adjusted for the various processes to minimize problems with sludge loadings, density currents, inlet hydraulic turbulence, and occasional poor sludge settleability. The size of the settling tank should be based on the larger surface area determined for surface loading rate and solids loading rate.

Design criteria for secondary clarifier is detailed in Table 4.7.

TABLE 4.7
SECONDARY CLARIFIER DESIGN CRITERIA

Treatment Process	Surface Loading Rate at Peak Design Flow[1] ($L/s/m^2$)	Peak Solids Loading Rate[2] ($kg/d/m^2$)	Minimum Water Depth (m)	Weir Overflow Rate[3] (L/s/m)
RBC Trickling filters	0.56	-	3.0	2.9 to 4.3
Conventional-activated Sludge; Contact Stabilization	0.56*	245	3.7**	2.9 to 4.3
Extended Aeration	0.47	171	3.7	2.9 to 4.3

1 Based on influent flow only. Lower loading rate should be used for nitrifying plants and those with chemical addition for phosphorus removal
2 Clarifier peak solids loading rate should be computed based on the maximum day design flow plus the maximum return sludge rate requirement and the design mixed liquor suspended solids (MLSS) under aeration.
3 Weir overflow rate would increase with increasing plant capacity.
* Plants needing to meet 20 mg/L suspended solids should reduce surface loading rate to 0.47 $L/s/m^2$.
** Greater water depths are recommended for clarifiers in excess of 372 m^2 surface area. Less than 3.7 m water depths may be adequate for package plants with average design flow less than 100 m^3/d.

4.3.1.9 Laboratory Requirements

All treatment works should include a laboratory for making the necessary analytical determination and operating control tests, except where satisfactory off-site laboratory provisions are made to meet the operating approval monitoring requirements. The laboratory should have sufficient size, adequate ventilation (particularly where furnace and fume hoods are used for solids and sludge analysis), bench space, equipment, and supplies to perform all self-monitoring analytical work required by the operating approval, and to perform the process control tests necessary for good management of each treatment process included in the design.

The laboratory arrangement should be sufficiently flexible to allow future expansion should more analytical work be needed. Laboratory instrumentation and size should reflect treatment plant size, staffing requirements, and process complexity. Experience and training of plant operators should also be assessed in determining treatment plant laboratory needs.

Before undertaking the detailed design of the laboratory facility, contact should be made with AESRD to confirm the testing requirements.

4.3.1.10 Flow Measurements

1. Location

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

   a. plant influent and effluent flow;

b. plant and process unit bypasses; and

c. other flows such as return activated sludge, waste activated sludge, recirculation, and recycle required for plant operational control.

2. Facilities

Indicating, totalizing, and recording flow measurement devices should be provided for all mechanical plants. All flow measurement equipment must be sized to function effectively over the full range of flows expected and shall be protected against freezing.

3. Hydraulic Conditions

Flow measurement equipment including entrance and discharge conduit configuration and critical control elevations should be designed to ensure that the required hydraulic conditions necessary for accurate measurement are provided. Conditions that must be avoided include turbulence, eddy currents, air entrainment, etc. that upset the normal hydraulic conditions that are necessary for accurate flow measurement.

4.3.1.11 Colour Codes

Refer to Table 4.8 for recommended colour coding for wastewater treatment plant piping.

TABLE 4.8
RECOMMENDED COLOUR CODING FOR WASTEWATER TREATMENT PLANT PIPING

Piping to be Identified	Basic Colour	Bands	
		No.	Colour
Raw Wastewater	Brown	-	-
Primary Settled Wastewater Effluent	Brown	1	White
Secondary Settled Wastewater Effluent	Gray	-	-
Sludge Lines			
Raw Wastewater	Black	-	-
Primary Sludge	Black	1	White
Secondary Sludge	Black	2	White
Digested Sludge	Black	3	White
Digested Liquor	Black	1	Brown
Natural Gas	Orange	-	-
Digester Gas	Orange	1	Black
Chlorine Gas	Yellow	-	-
Chlorine and Water	Pink	1	Yellow
Chlorinated Effluent	Grey	1	Yellow
Electrical	Purple	-	-
Compressed Air	White	-	-
Heating	Silver	-	-
Fire Protection	Red	-	-
Potable Water	Blue	-	-
Untreated Water	Dark Green	-	-

Notes
1. Entire length of pipe to be painted in basic colour.
2. Bands, if required, are to be placed as follows:
   (a) at 9 m intervals, and / or
   (b) where the pipe enters and leaves a room.
3. Individual bands are to be 25 mm wide, and a 25 mm space is to be left between bands where multiple bands are required.

4.3.2 Aerated Lagoons

4.3.2.1 General

The aerated lagoon system is a biological treatment process with long retention time (compared to mechanical systems) and large capacity. The system consists of one or more "complete mix cell" and one or more "aerated cell."

The complete mix cell is designed to provide enough oxygen transfer to satisfy the applied CBOD loading and to maintain a uniform solids concentration. The aerated cell is designed to satisfy the applied CBOD loading while maintaining an adequate uniform dissolved oxygen level in the cell. There is no attempt in the design of the aerated cell to provide complete solids mixing, therefore solids are allowed to settle in the cells to undergo anaerobic decomposition.

4.3.2.2 Design Approach

In general, the following factors should be considered in the design of the aerated lagoons:

- CBOD removal and effluent characteristics;
- Temperature effects;
- Mixing requirements;
- Oxygen requirements; and
- Solids separation.

1. CBOD Removal and Effluent Characteristics

   CBOD removal and the effluent characteristics are estimated using a complete mix hydraulic model and first order reaction kinetics. The complete mixed model using first order kinetics and operating in a series with 'n' equal volume cells is given by:

$$\frac{L_e}{L_i} = \frac{1}{\left[1 + \frac{K_t T}{n}\right]^n}$$

   where:

   L_e = Effluent BOD, mg/L
   L_i = Influent BOD, mg/L
   K_t = Reaction rate coefficient at t°C, day^{-1}
   T = Total hydraulic retention time in lagoon system, days
   n = Number of ponds in series

   The selection of the reaction rate coefficient is critical in the design of the lagoon system. All other considerations in the design will be influenced by this selection. If possible, a design K_{20} should be determined for the wastewater in pilot or bench scale tests; experiences of others with similar wastewaters and environmental conditions should also be evaluated. Reaction rate coefficient K_{20} may vary from 1.5 day^{-1} for complete mix cell to 0.37 day^{-1} for aerated cell.

   When using the complete mix model, the number of cells in series has a pronounced effect on the size of the aerated cell required to achieve a specific degree of treatment.

The reactor required to achieve a given efficiency may be greatly reduced by increasing the number of cells in series.

2. Temperature Effects

The influence of temperature on the reaction rate is expressed by the equation:

$$K_t = K_{20}\, \theta^{t-20}$$

where:

K_t = Reaction rate coefficient at t°C, day $^{-1}$
K_{20} = Reaction rate coefficient at 20°C, day $^{-1}$
t = Wastewater temperature, °C
θ = Temperature activity coefficient (varies from 1.04 to 1.1 for aerated lagoons, with typical value of 1.065)

3. Mixing Requirements

Aeration is used to mix the pond contents and to transfer oxygen to the liquid. There is no rational method available to predict the power input necessary to keep the solids suspended. The best approach is to consult equipment manufacturers' charts and tables to determine the power input needed to satisfy mixing requirements. Power of 6-10 w/m^3 of the cell volume is frequently used and these values can be used as a guide to make preliminary estimates of power requirements, but the final sizing of aeration equipment should be based on guaranteed performance by an equipment manufacturer.

For a complete mix cell, in comparing the power requirements for both, to maintain solids in suspension and to meet the oxygen demand, it would soon become evident that the mixing requirements would control the power input to the system.

After determining the total power requirements for a cell, the diffusers / aeration units should be located in the cell so that there is an overlap of the diameter of influence providing complete mixing.

4. Oxygen Requirements

Oxygen requirements are estimated using equations based upon mass balances, and there are several rational equations available to estimate the oxygen requirements for lagoon systems, however, the use of the CBOD entering the pond as a basis to estimate the biological oxygen requirements, is as effective. Approximately 1.5 kg to 2.0 kg of oxygen is required to remove 1 kg of CBOD in the aerated lagoon system.

As for the other mechanical systems, monitoring of aerated lagoons include adjustment of aeration devices to control dissolved oxygen to be greater than 2 mg/L in the aeration basin during peak loading conditions. Lagoons with odour problems should be corrected by increasing the air supply.

5. Solids Separation

For systems with continuous discharge to a receiving stream, a polishing cell having a minimum hydraulic retention of five days, based on summer average daily design flows, should be provided. Polishing cells are not required for systems having storage facilities with intermittent discharges.

4.3.2.3 System Design (Aeration)

The preceding section outlined the general design approach for designing aerated lagoon systems. This section outlines the procedure for the determining the aeration requirement for mixing (Step 1) as well as for satisfying the applied CBOD loading (Steps 2 to 6) in a step format.

Step 1: Determine aeration requirement for mixing in the complete mix cell. As indicated earlier, a power input of 6 - 10 w/m^3 per cell volume may be assumed to make the preliminary estimates, however the final capacity should be based on guaranteed performance by an equipment manufacturer.

Step 2 - Calculate the CBOD reduction in each cell under both summer and winter conditions.

Modifying the formula under section 4.3.2.2 (1) for a single cell, CBOD reduction (complete mix cell) as a % of the average daily design CBOD load (E_1);

$$E_1 = \frac{K_tT}{1 + K_tT} \times 100$$

CBOD reduction (aerated cell) as a % of the average daily design CBOD load (E_2);

$$E_2 = \frac{K_t}{1 + K_tT} \times E_1 \times 100$$

CBOD reduction (overall) $E_3 = E_1 + E_2$

K_t values can be determined utilizing the formula noted in section 5.3.2.2(2).

Insert the calculated % reduction values on line 1 in the summary Table 5.8.

Step 3 - Calculate the CBOD removal (Kg) in each cell during both summer and winter conditions as follows:

CBOD removed in complete mix cell = average daily design CBOD load x E_1 (summer and winter).

CBOD removed in Aerated Cell = average daily design CBOD load x E_2 (summer and winter).

Insert the calculated CBOD removed in each cell on line 2 in the summary Table 4.8 and complete line 3.

Step 4 - Calculate the Actual Oxygen Required (AOR) in Kg/hour. This is determined by multiplying the calculated CBOD removal rate as noted on line 3 in the summary table by the oxygen to CBOD ratio (oxygen required in Kg per Kg of CBOD removed). As indicated in the foreceding section, an oxygen to CBOD ratio of 1.5 to 2.0 could be used when designing a system handling typical domestic wastewater.

Insert the AOR on line 4 in the summary Table 4.8.

Step 5 - Calculate the Standard Oxygen Required (SOR) in Kg/hour for each cell and for the total system during summer and winter conditions. This requires determination of the oxygen mass transfer ratio (φ) which is utilized to correct the rate of oxygen transfer under standard conditions to the rate of oxygen transfer to the wastewater under site conditions. The formula for determining the oxygen mass transfer ratio is as follows:

$$\varphi = \frac{\alpha(\beta C_s - C_L)\gamma^{t-20}}{C_s}$$

where:

φ = oxygen mass transfer ratio

α = correction factor used to estimate the actual oxygen transfer in wastewater versus the oxygen transfer in low total dissolved solids water generally used for rating aeration devices. α values of 0.6 to 0.8 are generally used for design.

β = correction factor used to correct the test system oxygen transfer rate for differences in oxygen solubility due to constituents in the wastewater such as salts, particulates and surface active substances. Values range from 0.7 to 0.98. A value of 0.95 is commonly used in the absence of experimental verification.

γ = factor for correcting the oxygen mass transfer rate at temperatures other than 20°C. Typical values are in the range of 1.015 to 1.040. A value of 1.024 is typical for both diffused and mechanical aeration devices.

C_s = solubility of oxygen at site conditions (i.e., site temperatures and barometric pressure) - mg/l.

C_L = minimum required dissolved oxygen concentration in the treated effluent - (2 mg/L during peak loading conditions).

t = wastewater temperature °C

Step 6 - Calculate the SOR required in Kg/hr for each cell and for the total system under both winter and summer conditions as follows:

SOR for each cell = AOR/φ

Total SOR = sum of the SOR for all cells

Insert the SOR values on line 6 in the summary Table 4.9.

TABLE 4.9
AERATION SYSTEM DESIGN
SUMMARY TABLE

Line No.	Summer			Winter		
	Mix Cell	**Aerated Cell(*s*)**	**Total**	**Mix Cell**	**Aerated Cell(*s*)**	**Total**
1. CBOD reduction as a % of average daily design BOD loading.						
2. CBOD removed - Kg/d						
3. CBOD removed - Kg/hr.						
4. AOR - Kg/hr.						
5. φ- oxygen transfer correction ratio.						
6. SOR - Kg/hr.						

The highest total oxygen transfer rate is used to size the aeration system for the applied CBOD loading.

4.3.3 Odour Control

4.3.3.1 Odour Production

1. Odour Development

Wastewater contains numerous potentially odorous substances, but the predominant group are the reduced sulphur compounds. Of these, hydrogen sulphide is perhaps the most common and the most easily identified. For this reason, odour control measures concentrate on sulphide control.

There are several texts which discuss odour generation, particularly as related to sulphides. The designer is referred to the U.S. EPA Design Manual, “Odour and Corrosion Control in Sanitary Sewage Systems and Treatment Plants” (EPA/625/1-85/018), and the ASCE Manual of Practice No. 69, “Sulfide in Wastewater Collection and Treatment Systems”.

2. Odour Measurement and Limits

Odour measurement is largely subjective. The most commonly accepted method of characterization is the ‘Odour Unit’ (OU). The OU is based on the number of dilutions with clean air required to reach a threshold detection level. OU values are presented as an odour sample’s ‘Effective Dose’ - 50th percentile (ED_{50}), meaning the number of dilutions at which an odour is detected by half the members of an odour panel using a dynamic dilution olfactometer. Thus a sample which requires four dilutions to reach ED_{50} will contain 5 OU (four dilutions plus the original volume).

The designer must determine, in consultation with the Owner and AESRD (and often the public), the appropriate odour limits and how they should be applied to the facility. Often a 'fenceline' odour limit is applied, which determines the magnitude of the odours acceptable at the boundary of the facility. The limit will depend on the proximity of residential and commercial development, and other site-specific factors such as the proximity of parks, trails or roads and the sensitivity of the odour problem.

4.3.3.2 Potential Odour Sources

Odour problems tend to develop when dissolved sulphide concentrations exceed 0.5 mg/L, or less if pH is depressed. Sulphide production commences in the collection system, and will continue to occur wherever anaerobic deposits accumulate. The rate of sulphide production and odour generation are both temperature dependent.

Industrial discharges frequently exacerbate odour. Some discharges have high sulphide contents; others may have a low pH or high temperature.

Turbulence promotes sulphide stripping and hence odours. In the collection system, this occurs at drop manholes, sharp bends, forcemain discharge points and any hydraulic structure where turbulence or super-critical flow develops.

Generally, the odour-emission potential at treatment plants decreases at each successive treatment stage. The influent sewer and headworks receive sewage with higher sulphide content and are often turbulent areas. Preliminary treatment processes can generate odours from the screenings and grit handling areas. Aerated grit tanks will also strip sulphides.

Further sulphide generation often occurs as a result of anaerobic action in the sludge blankets accumulating in primary sedimentation tanks. The resultant hydrogen sulphide can be stripped at the effluent weirs due to the turbulence developed there.

Aeration basins do not usually generate high sulphide odours unless overloaded, as sulphides are oxidized within the basin. Attached growth systems are more likely to generate odours, particularly if the growth becomes excessive. Final clarifiers rarely produce significant odours unless there are problems with the sludge or scum handling systems.

Solids handling and treatment processes have significant odour generation potential because of the high sulphide concentrations present in sludge, scum and septage. Aerobic digesters, thickening and dewatering processes and sludge storage lagoons are all potential odour sources.

4.3.3.3 Evaluation of Odour Production Potential

1. Monitoring Protocols

   Detailed monitoring exercises should be preceded by a preliminary study to analyze available data and odour complaints. Complaints should be correlated with data on plant operations and wastewater characteristics and meteorological data. The preliminary study may include limited on-site sampling and analyses.

   Detailed field monitoring programs should be of sufficient duration to monitor seasonal variations in sulphide generation and Hydrogen Sulphide emissions. Monitoring should also include an hourly sampling and testing regime to identify typical diurnal fluctuations.

   Sampling points should be readily identifiable and remain consistent throughout the monitoring program.

   a. Liquid Phase Analyses

      Routine parameters to be monitored should include total and dissolved sulphides, CBOD or COD, temperature, pH and dissolved oxygen (D.O.). Oxidation-reduction potential (ORP) can also yield useful data.

      Additional analyses for TSS and particle size distribution will be required if the results are to be used for predictive modelling of sulphide generation.

   b. Gas Phase Analyses

      In-situ gas phase testing can be used to identify a wide range of odorants, including Hydrogen Sulphide, Mercaptans and Dimethyl Disulphide. Continuous monitoring may be necessary in some cases to identify the peak Hydrogen Sulphide gas concentrations which trigger odour complaints. The designer should ensure that the equipment to be used for gas phase testing is suitable for the range of concentrations expected.

   c. Air Sampling

      Foul air sampling will be required if it is intended to use a dynamic dilution olfactometer and odour panel to determine OU values. Specialized sampling equipment and sample bags will be required.

   d. Gas Chromatography

      GC analyses are useful for identifying total levels of sulphides, and other potential odorants. Analyses can be carried out on liquid or gas phase samples. The collection and preservation of samples shall be in accordance with established procedures for these type of analyses.

2. Interpretation of Results

Data obtained from monitoring programs will provide the designer with useful information concerning the conditions governing sulphide generation and Hydrogen Sulphide release.

Areas of anaerobic activity producing sulphides will be characterized by low D.O. (less than 0.5 mg/L) and negative ORP. Reaction kinetics will be temperature dependent. The rate of sulphide generation will be greater in the presence of higher fractions of soluble BOD.

Hydrogen sulphide emissions will increase at lower pH and higher temperatures.

3. Predictive Modelling

a. Sulphide Generation in Sewers and Forcemains

A number of predictive models have been developed for this purpose. The models have been developed from empirical data and as such are valid only for specific conditions. The designer is cautioned to ensure that the chosen method of analysis is applicable under the conditions in question.

b. Air Quality Computer Modelling

If designing to specific odour limits at identified receptor points, the designer should consider the use of a computer based atmospheric dispersion model to simulate the behaviour of the odour plume.

4.3.3.4 Odour Control and Abatement Measures

It is generally more reliable and cost effective to treat and remove odorants in the liquid phase rather than collecting and treating foul air. Sulphides develop in the collection system and the designer should therefore consider upstream control measures in conjunction with measures at the wastewater treatment plant. Such measures should include designing to prevent deposition in sewers, minimizing residence time in pump station sumps, and avoiding the use of siphons and long forcemains.

1. Prevention of Sulphide Formation

For new treatment facilities, the design should attempt to eliminate ‘dead zones’ where solids may accumulate.

Fillets should be incorporated into rectangular channels, conduits and tanks. Inverted siphons should be avoided.

Where self-cleaning velocities cannot be achieved over the full flow range, aeration should be provided to channels and conduits. Excessive aeration should be avoided because of potential odour generation due to increased turbulence. Provide sufficient energy input per unit volume to ensure solids are maintained in suspension.

The designer should also consider improved access provisions to facilitate routine housekeeping and cleaning activities.

2. Chemical Treatment

When dosing chemicals into sewage, side reactions will occur in addition to the desired reaction. In calculating dosing rates, the designer should allow a generous factor of safety to account for these side reactions. Pilot testing is recommended for all chemical dosing systems.

a. Oxidizing Agents

Chlorine (as gas or sodium hypochlorite), potassium permanganate and hydrogen peroxide will oxidize sulphides and inhibit sulphide production. Pure oxygen and air injection have also been used to raise D.O. levels in the sewage.

b. Precipitants

Iron and zinc salts will precipitate sulphides. Ferrous and Ferric Chloride have been used extensively, in collection systems, forcemains and at wastewater treatment plants. The designer should consider the effect on the solids handling streams at the wastewater treatment plant, in terms of increased sludge production and increased levels of contaminants in the sludge.

c. pH Control

Intermittent slug dosing with sodium hydroxide will raise the pH, inhibiting sulphide production and preventing hydrogen sulphide off-gassing. This system is effective only in localized areas and should be considered only for specific trouble spots in the collection system.

d. Electron Acceptors

Electron acceptors are taken up preferentially to the sulphate ion, and thus prevent sulphide formation. Sodium nitrate has been used in lagoons for this purpose. Proprietary nitrate products have also been used in sewers.

3. Control of Mass Transfer

The transfer of sulphides from liquid to gas phase can be reduced by minimizing liquid turbulence and reducing aeration. The designer should consider the following measures to reduce turbulence:

a. Minimize elevation differences where streams converge;

b. Introduce side streams below the liquid surface;

c. Use of submerged effluent weirs and downstream flow control in lieu of conventional launders for sedimentation tanks and clarifiers;

d. Avoid excessive or unnecessary aeration;

e. Avoid the use of screw lift pumps on potentially odorous streams.

4. Foul Air Collection and Treatment

Emission control systems should be considered for all solids handling areas and processes, and for other areas of the facility where preventative measures are insufficient to mitigate odours.

a. Covers

Cover systems should be designed to minimize the number of joints. Seals should be provided at all joints. The designer should consider the corrosive action of sulphides and sulphuric acid when selecting cover materials and concrete coatings. Overhangs, ledges or lips on the underside of covers where condensate may collect should be avoided.

The design of covers should be aimed at minimizing the volume of air requiring treatment.

The designer should consider operational requirements for access and cover removal, and be aware that the installation of covers will create a confined space environment.

b. Ventilation

Ventilation rates should be based on the more stringent of two requirements: (a) maintain a slight negative pressure in the headspace and thus prevent fugitive odours escaping through joints in the cover system; and (b) limit sulphide concentrations in the air stream to a level which the downstream treatment systems can effectively treat.

c. Treatment Systems

There are numerous alternative treatment systems available. Selection of the appropriate system should be subject to a life cycle cost-benefit analysis. The systems that may be considered include:

i. chemical scrubbers, packed bed or mist contactor types;

ii. activated carbon, with or without chemical impregnation;

iii. activated alumina impregnated with potassium permanganate;

iv. biofilter, in-vessel or soil / compost types;

v. incineration;

vi. dual stage systems comprising one or more of the above.

Design requirements for these systems vary considerably. The designer should consult equipment manufacturers for details.

When assessing bulk chemical storage requirements, the maximum effective storage life of the chemical should be considered. Many chemicals are temperature-sensitive and storage tanks will require special provisions if located outdoors.

Provision will also be required for disposal of waste and side-streams from the treatment systems, which may require further treatment before return to the main liquid stream at the wastewater treatment plant.

An alternative form of biological treatment or pre-treatment involves blowing foul air through aeration basins or trickling filters. If this form of treatment is considered, the designer should be aware of the potential for corrosion in the air blowers and aeration system.

4.3.4 Instrumentation and Controls

4.3.4.1 General

Several factors should be considered when developing a plan for the instrumentation and controls for a wastewater treatment facility. Alberta Environment and Sustainable Resource Development 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:

- plant size and complexity
- regulatory requirements
- hours of attended operation
- potential chemical and energy savings
- primary element reliability
- primary element location
- whether controls should be manual **or** automatic
- 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:

- flow rate for raw sewage, by-pass flows, final effluent flow
- return Activated Sludge (RAS) flows, Waste Activated Sludge (WAS) flows

- raw and digested sludge flow, digester supernatant flows
- chemical dosage, digester gas production
- hazardous gas monitoring
- anaerobic digester temperature

- dissolved oxygen levels
- sludge blanket levels and sludge concentrations.

4.3.4.2 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.

1. Flow Measurement

   a. Magnetic Flow meters (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 (1-10 m/s) for non-solids bearing liquids and (1.5-7.5 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.

   b. Ultrasonic Flow meters

   Flow meter Construction. The flow meter 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 (1-10 m/s).

Applications. Transmittance style is not recommended for influent wastewater, primary sludge, thickened sludge, nitrification RAS, or nitrification WAS. Reflective style is not recommended for primary effluent, secondary clarifier effluent final effluent or process wash water.

c. Turbine Flow meters

Flow meter Construction. The flow meter 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.

d. 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.

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.

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.

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 0-20-mg/L.

5. Level Measurement

a. 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.

b. 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.

c. 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. Pressure Measurement

a. 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 0-35000 kPa.

b. 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 0-2000 kPa.

c. 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 0-3500 kPa.

7. Temperature Measurement

a. 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.

b. 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 0-300 C.

c. 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 0-300 C.

d. Thermal Bulb

Installation. No special installation requirements.

Applications. Thermal bulbs are suitable for temperature measurement applications with ranges of 0-500 C.

4.3.4.3 Process Controls

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.

   a. 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 level elements.

      When variable speed pumps are used there are several ways in which the pump can be controlled. These generally centre around control to maintain a level set point in the wet well. This requires a feedback type of control in which the measured variable (level) is compared to a set point value and the final control element is modulated 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.

   b. 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. [See also Section 4.2.4.2 (5)].

   c. 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.

   d. Pumps and Motors

      The following parameters should be monitored:

      - pump bearing temperature;
      - pump bearing vibration;

- pump speed for variable speed applications;
- pump discharge pressure;
- motor voltage and current;
- motor hours of operation;

- motor bearing temperature; and
- motor windings temperature.

e. Alarms

Lift stations should be alarmed as outlined in Section 4.2.4.5.

2. Mechanical Bar Screens

Three methods are used to control the operation of mechanical bar screens:

a. Simple manual start / stop which requires the presence of the operator at the screen in order to start and stop the screen.

b. 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.

c. 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.

3. Primary Treatment

a. Raw Sludge Pumping

The raw sludge pumping should be set up to incorporate the following features:

- automatic or manual selection of duty pump;
- on-line sludge density metering for control and monitoring;
- on-line sludge flow monitoring and totalization;
- on-line adjustable sludge density control;
- individually selectable hopper pumping controls where required;

- manual override for automatic controls;
- on-line sludge blanket level monitoring and alarming;
- on-line sludge pump monitoring and control;
- sludge density feedback control for variable speed pumping with manual override;
- on-line sludge pump speed monitoring and control with manual override; and
- on-line monitoring and control of primary tank scraper mechanisms.

b. Scum Pumping

The scum pumping should be set up to incorporate the following features:

- automatic or manual selection of duty pump;
- manual override for automatic controls;
- on-line sludge blanket level monitoring and alarming;
- automatic controls consisting of high and low scum tank level for starting and stopping scum pumps;
- high scum tank level alarm;
- on-line scum pump speed monitoring and control with manual override; and
- scum tank flushing system for scum tank cleaning.

4. Secondary Treatment

a. Dissolved Oxygen (D.O.) Control

Automatic D.O. control systems should be used to control the rate of air supply to aeration tanks. The following methods may be used:

i. 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 airflow to the aeration tanks. Automatic dissolved oxygen control should always be equipped with manual override.

ii. Feed Forward Control

Feed forward control consists of a fixed volume of air being delivered to the aeration tanks for a given flow rate. This system may utilize on line dissolved oxygen analyzers but these are used for monitoring only and do not provide feedback to the airflow control elements.

Process status and alarms should be provided for dissolved oxygen level, blower operating parameters, airflow control elements.

b. Return Activated Sludge Control

The Return Activated Sludge pumping should be set up to incorporate the following features:

- automatic or manual selection of duty pump;
- variable speed pumping;
- Return Activated Sludge flow monitoring;
- feedback control to match pumping rates to flow set points;
- individual control of sludge return rate from individual final clarifiers;
- manual override for automatic controls; and
- on-line monitoring of Return Sludge flow rate, pump speed and status.

c. Waste Activated Sludge Control

The Waste Activated Sludge pumping should be set up to incorporate the following features:

- automatic or manual selection of duty pumps;
- variable speed pumping;
- Waste Activated Sludge flow monitoring;
- feedback control to match pumping rates to flow set points;
- manual override for automatic controls; and
- on-line monitoring of Waste Sludge flow rate, pump speed and status.

d. 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 flow rate. Chemical dosing requirements will vary widely depending on performance requirements and the specific process being utilized.

e. 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.

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.

a. 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.

b. 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 (I/O) cards located in what are called racks. The racks may be mounted separately or placed in specific plant areas to reduce wiring costs. The I/O 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.

4.3.4.4 Design Documents

Complete design documents should be prepared to ensure that construction can be completed correctly and also to properly record the system for future reference. The following are required in the design documents:

- design and construction standards, specifications and installation details
- panel sizing and general arrangements
- control system functional requirements
- control component and instrument data sheets
- operator interface and control hardware and software specifications including input and output (I/O) lists
- control system programming and packaged system configuration standards, structure and scope.

4.3.4.5 Control System Documentation

The following documents should be provided following completion of the control system:

- record drawings to show any changes to the design and including any drawings produced during construction
- annotated listings of control system programs and packaged system configuration
- manufacturer's literature for all control and instrumentation components
- final wiring diagrams complete with wire and terminal coding
- motor control schematics
- instrument loop diagrams
- panel wiring and layout details
- PLC or DCS wiring schematics
- instrument calibration sheets
- operating instructions.

4.3.4.6 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.

4.3.5 Emergency Facilities and Component Reliability

4.3.5.1 Back-Up Requirements

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

This does not mean that the tankage capacity has to be doubled. To achieve this, critical treatment processes should be provided in multiple units so that with any unit out of operation, the hydraulic capacity of the remaining units should be sufficient to handle the peak wastewater design flow. There should also be sufficient flexibility in capability of operation so that the normal flow into a unit out of operation can be distributed to all the remaining units.

With some processes such as mechanical screening, the backup can be provided with a less sophisticated unit such as a manually cleaned screen.

Sewage and sludge pumping units should be designed such that with any unit out of service, the remaining pumps operating in parallel should be capable of pumping the peak flows. In certain instances, particularly with sludge pumps, one pump may serve as a back-up for more than one set of pumps, i.e. a raw sludge pump could back-up a sludge transfer pump, etc. Standby capacity requirements for sludge return pumps should be determined on a case-by-case basis.

Aeration systems will require facilities to permit continuous operation, or minimal disruption, in the event of equipment failure. The following factors should be considered when designing the back-up requirements for aeration systems:

- effect on the aeration capacity if a piece of equipment breaks down, or requires maintenance (for instance, the breakdown of one of two blowers will have a greater effect on capacity than the breakdown of one of four mechanical aerators);

- time required to perform the necessary repair and maintenance operations;

- the general availability of spare parts and the time required to obtain delivery and installation.

Generally, considerations such as the above will mean that diffused aeration systems will require a standby blower (maximum air demand should be met with the largest blower out-of-service), but mechanical aeration systems may not require standby units, depending upon the number of duty units, availability of replacement parts, etc.

Chemical feed equipment (e.g. phosphorus removal and disinfection) should be provided in multiple units so that the chemical requirements can be supplied with one unit out of operation.

With 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. In smaller plants, multiple primary and secondary digestion units can often be avoided by this method. When such an approach is proposed, the designer should outline the alternate methods of treatment and disposal that could be used during periods of equipment breakdown. With larger treatment plants, the provision of multiple primary and secondary digestion units can usually be economically justified. Single stage digesters will generally not be satisfactory due to the usual need for sludge storage, and effective supernating.

If effluent filtration is employed, Provision of multiple effluent filtration units may be necessary, depending upon the receiving stream sensitivity, type of filtration equipment, and the maintenance requirements of the filter units.

With sludge handling and 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. 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.

4.3.5.2 Wastewater Bypass Facilities

To allow maintenance operations to be carried out, each unit process within the treatment plant should be provided with a bypass facility around the unit.

Where two or more similar treatment units are considered and one unit is out of operation for repairs, the remaining units should be capable of passing the peak wastewater design flow rates or be provided with bypass capacity equal to the excess hydraulic flow of the operating units.

Bypass systems should also be constructed so that each unit process can be separately bypassed.

All flows bypassing secondary and / or tertiary treatment processes should be measured.

4.3.5.3 Standby Power

1. General

   Plants should be provided with an alternate source of electric power or pumping capability to allow continuity of operation during power failures, except as noted below. Methods of providing alternate sources include:

   a. the connection of at least two independent power sources such as substations. A power line from each substation should be installed;

   b. portable or in-place internal combustion engine equipment which will generate electrical or mechanical energy; and

   c. portable pumping equipment when only emergency pumping is required.

2. 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 (four hours or more) power outages have occurred, auxiliary power for minimum aeration of the activated sludge may be provided.

3. Power for Disinfection

   When receiving water stream is environmentally sensitive, continuous disinfection should be provided during all power outages.

4.4 Residual / Biosolids (Sludge) Processing

4.4.1 General

The treatment, handling and disposal of wastewater sludges should be integrated with the planning and design of all wastewater treatment plants. The purpose of sludge processing is to reduce and stabilize biodegradable organic matter so that handling and disposal may be done in an environmentally acceptable manner. Techniques for processing and disposing of sludges will depend on characteristics of the wastewaters and sludges, the wastewater treatment process, and the size and location of the wastewater treatment facility.

Facilities for processing sludge should be provided at all mechanical wastewater treatment plants. Handling equipment should be capable of processing sludge to a form suitable for ultimate disposal. If ultimate disposal method is not suitable year round, provision must be made for sludge storage during the period disposal is not possible.

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. sludge digestion or stabilization requirements;
5. sludge storage requirements; and

6. methods of ultimate disposal.

4.4.2 Biosolids (Sludge) Handling and Treatment

4.4.2.1 Digestion

Sludge stabilization is generally achieved by digestion. Two types of digestion systems are used - anaerobic and aerobic.

Anaerobic digestion is the most commonly used system for the digestion of primary and mixtures of primary and waste activated sludges. Aerobic digestion, because of the relatively large energy requirements, is not recommended for use, particularly at large wastewater treatment facilities.

1. Anaerobic Digestion

   Anaerobic sludge digestion is used to reduce and stabilize the biodegradable organic matter to improve the dewatering characteristics of sludge and to reduce pathogenic organisms. Bulky, odorous raw sludges are converted to a relatively inert material that can be readily dewatered in the absence of offensive odours. Oxygen is excluded from the anaerobic digestion process, and most pathogenic organisms are destroyed by properly designed digester tanks operating at temperatures around 35°C over a period of about 15 to 30 days. Temperature, pH, mixing, and retention time are the critical design factors which should be considered.

   Anaerobic digesters should meet the requirements of the latest edition of "CAN/CGA-B105, Code for Digester Gas and Landfill Gas Stations. Digestion systems should also be designed with features and in accordance with design parameters, as follows:

   - *Number of Stages*

     Two (primary and secondary).

   - *Number of Digesters in Each Stage*

     One adequate in small plants provided that flexibility is given to allow either stage to receive raw sludge in emergencies; number of digesters in each stage of large plants will be dictated by economics.

   - *Hydraulic Retention Time in Primary Digester*

     Minimum 15 days (sludge retention time requirements of slowest methane producers is approximately 10 days).

   - *Mixing*

     For digestion systems utilizing two stages, the first stage (primary) should be completely mixed (via digester gas - compressor power requirements 5 to 8 W/m^3 or mechanical means - 6.6 W/m^3). The second stage (secondary) is to be designed for sludge storage, concentration, and gas collection and should not be credited in the calculations for volumes required for sludge digestion.

- *Volatile Solids Loading*

  0.5 - 1.5 kg/m^3/d.

- *Completely Mixed Systems*

  For digestion systems providing for intimate and effective mixing of the digester contents, the system may be loaded with volatile solids up to 1.3 kg/m^3 of volume per day in the active digestion units.

- *Moderately Mixed Systems*

  For digestion systems where mixing is accomplished only by circulating sludge through an external heat exchanger, the system may be loaded with volatile solids up to 0.65 kg/m^3 of volume per day in the active digestion units. This loading may be modified upward or downward depending upon the degree of mixing provided.

- *Heating*

  Heating must be provided for the primary digester so that a temperature of 35^{o}C can be maintained. External heat exchanger systems are preferred. Heating should be via a dual-fuel boiler system using digester gas and natural gas, or oil.

- *Digester Covers*

  Digester covers may be fixed or floating type, sized to provide gas storage volume. Insulated pressure and vacuum relief values and flame traps should be provided. Access manholes and sampling wells should also be provided in the digesters covers. The underside of the roof must be protected from corrosion. Coatings should be compatible with the roofing material and the environment in the digester to ensure a good bond.

  Steel, concrete or fibreglass covers may be used.

- *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. Off-site storage in sludge lagoons, sludge storage tanks, or other facilities, may be used to supplement the storage capacity of the secondary digester.

- *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; minimum diameter of sludge pipes should be 100 mm; 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.

- *Supernatant Piping*

  Supernatant should be returned to the treatment plant with flexible points of return to the grit removal facilities, upstream of the primary settling tanks, or to the aeration tank; multiple draw-off points or adjustable supernatant draw-offs, and sampling points should be provided; both primary and secondary digesters should be equipped with supernatant piping so that during emergencies the primary can be operated as a single stage process; additional $CBOD_5$ load caused by supernatant return should be considered in aeration system design.

- *Overflows*

  Each digester should be equipped with an emergency overflow system.

- *Waste Gas*

  a. Location

     Waste gas burners shall be readily accessible and should be located at least 15 m away from any plant structure. Waste gas burners shall be of sufficient height and so located to prevent injury to personnel due to wind or downdraft conditions.

  b. Pilot Light

     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.

  c. Gas Piping Slope

     Gas piping shall be sloped at a minimum of 2 percent up to the waste gas burner with a condensate trap provided in a location not subject to freezing.

2. Aerobic Digestion

   Aerobic digesters treating waste activated sludge should be designed in accordance with the following criteria. If primary sludge is to be included, minimum sludge age and air requirements may have to be increased.

   - *Number of Stages*

     Two.

   - *Number of Tanks in Each Stage*

     Generally one.

   - *Loading*

     1.5 $kg/m^3.d$ volatile suspended solids based upon first stage volume only.

- *Sizing*

  Designed to achieve a minimum sludge age of 45 days, including both stages and sludge age of waste activated sludge; if a total of 45 days sludge age is all that is provided, it is suggested that 2/3 of the total digester volume be in the first stage and 1/3 be in the second stage; 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.

- *Air and Mixing Requirements*

  Aeration rate will depend upon the oxygen uptake rate at the maximum solids content experienced; as a guideline, 0.85 L/m^3.s (litres of air per cubic metre of aeration tank per second) should be provided for diffused aeration systems; a minimum bottom velocity of 0.25 m/s should be maintained while aerating; mechanical surface aeration systems are not recommended due to increased heat loss causing icing problems.

- *Tank Design*

  Generally open; tankage should be of common wall construction or earthern-bermed to minimize heat loss; tank depths 3.6-4.6 m; tanks and piping should be designed to permit sludge addition, sludge withdrawal, and supernatant decanting from various depths to, or from both the primary and secondary digester.

4.4.2.2 Conditioning

Sludge dewatering, and to a lesser extent sludge thickening operations, are highly dependent upon sludge conditioning for their successful operation. Sludge conditioning not only affects the solids concentration of the thickened or dewatered sludge, but also affects the solids capture efficiency of the process.

There are two sludge conditioning approaches that can be used. Sludge can be conditioned by physical methods, such as heat treatment, or by chemical methods, involving the addition of either organic or inorganic chemicals.

The method selected will not only differ in its effect on the thickening or dewatering process, but will have different effects on subsequent sludges handling operations and on the sewage treatment process itself.

1. Physical Methods

   Heat conditioning of sludge consists of subjecting the sludge to high levels of heat and pressure. With this process, the sludge is treated at temperatures of 175 to 204^{o}C, pressure of 1700 to 2800 kPa and for detention times of 15 to 40 minutes. The high temperatures cause hydrolysis of the water-solids matrix and breaking down 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.

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 the supernatant is returned to the aeration system, due to the solubilization of organic matter during the sludge hydrolysis. This liquor can represent 25 to 50 percent of the total loading on the aeration tanks and allowances must be made in the treatment plant design to accommodate this loading increase.

Heat conditioning results in the production of extremely corrosive liquids requiring the use of corrosion-resistant materials such as stainless steel. Scale formation in the heat exchangers, pipes and reactor is a common problem.

The design requirements for a heat conditioning system should be determined by either batch or small-scale continuous pilot plants. Through such methods, the necessary level of hydrolysis to produce the desired reduction in the specific resistance of the sludge, and the liquor characteristics can be determined. Tests can also be made at different temperatures and retention times to determine the most effective full-scale operating conditions.

Freezing of sludges has been used successfully for water treatment plant sludges, but not common as a conditioning method for sewage sludges.

2. Chemical Methods

Chemical conditioning methods involve the use of organic or inorganic flocculants to promote the formation of a porous, free draining cake structure. Chemical conditioning for thickening operations attempts to promote more rapid phase separation, higher solids concentration and a greater degree of solids capture. With dewatering operations, chemical conditioning is used in an attempt to enhance the degree of solids capture by destabilization and agglomeration of fine particles. This promotes the formation of a cake which then becomes the true filter media in the dewatering process.

With most thickening operations and with belt filter press dewatering operations the most commonly used chemicals are high molecular weight polymers. The selection of the most suitable chemical(s) and the dosage requirements for sludge conditioning can be best determined initially by bench and pilot testing.

Laboratory testing should, however, be used to narrow down the selection process and to arrive at approximate dosage requirements. Generally, laboratory testing will yield dosage requirements within 15 percent of full-scale needs.

4.4.2.3 Thickening

Sludge thickening can be employed in the following locations in a wastewater treatment plant:

- prior to digestion for raw primary, excess activated sludge or mixed sludges;
- prior to dewatering facilities;
- following digestion for sludges or supernatant;
- following dewatering facilities for concentration of filtrate, decant, centrate, etc.

The commonly employed methods of sludge thickening are gravity and air flotation. Their suitability for the various types of sludge are shown in Table 5.10. Centrifuges, gravity belt thickeners and rotating drum thickeners are also used for sludge thickening. All thickening devices are adversely affected by high sludge volume indexes (SVIs) and benefited by low SVIs in the influent activated sludges. The ranges of thickened sludge concentrations given in Table 4.9 assume an SVI of approximately 100.

Wherever thickening devices are being installed special consideration should be given to the need for sludge pre-treatment in the form of sludge grinding to avoid plugging pumps, lines, and thickening equipment. Also, where thickeners are to be housed, adequate ventilation should be provided.

1. Gravity

   Gravity thickening is principally used for primary sludge, and mixtures of primary and waste activated sludges, with little use for waste activated sludges alone.

   Gravity thickeners should be designed in accordance with the following parameters:

   - *Tank Shape*

     Circular.

   - *Tank Depth*

     3 to 3.7 m.

   - *Tank Diameter*

     Up to 21 - 24 m.

   - *Floor Slope*

     Acceptable range 2:12 to 3:12.

   - *Solids Loading*

     Primary sludges 96 to 120 $kg/m^2.d$; waste activated 12 to 36 $kg/m^2/.d$; combination of primary and waste activated based on weighted average of above loading rates.

   - *Overflow Rate*

     0.19 to 0.38 $L/m^2.s$.

   - *Chemical Conditioning*

     Provision should be made for the addition of conditioning chemicals into the sludge influent lines.

- *Sludge Volume Ratio*

  Volume of sludge blanket divided by volume of sludge withdrawn daily should be 0.5 to 2 days.

TABLE 4.10
SLUDGE THICKENING METHODS AND PERFORMANCE WITH VARIOUS SLUDGE TYPES

Thickening Method	Sludge Type	Performance Expected
GRAVITY	Raw Primary	Good, 8 to 10% Solids
	Raw Primary and Waste Activated	Poor, 5 to 8% Solids
	Waste Activated	Very Poor, 2 to 3% Solids (Better results reported for oxygen excess activated sludge)
	Digested Primary	Very Good, 8 to 14% Solids
DISSOLVED AIR FLOTATION	Waste Activated (Not generally used for other sludge types)	Good, 4 to 6% Solids and _ 95% Solids Capture

2. Air Flotation

   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.

   Flotation operations cannot be designed on the basis of purely mathematical formulations or by the use of generalized design parameters and some bench-scale and / or pilot-scale testing will be necessary. The following design parameters are given only as a guide to indicate the normal range of values experienced in full-scale operation:

- *Tank Dimensions*

  Vary with suppliers.

- *Air Buoyancy Systems*

  Vary with suppliers.

- *Air to Solids Weight Ratio*

  0.02 to 0.05.

- *Recycle Ratios*

  Vary with suppliers (0 to 500%).

- *Solids Loadings (with waste activated sludge to achieve 5% float solids)*

  48 $kg/m^2.d$ (without flocculating chemicals); up to 240 $kg/m^2.d$ (with flocculating chemicals).

- *Chemical Conditioning*

  Feed chemical to mixing zone of sludge and recycled flow.

- *Hydraulic Feed*

  Up to 1.74 $L/m^2/.s$ (based on total flow including recycle, when polymers used); without chemicals, lower rate must be used; feed rate should be continuous.

- *Detention Time*

  Not critical provided particle rise rate is sufficient and horizontal velocity in the unit does not produce scouring of the sludge blanket.

4.4.2.4 Dewatering

Sludge dewatering will generally be required prior to ultimate disposal of sludges, other than for land application. 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 wastewater 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.

Table 4.11 gives the solids capture, solids concentrations normally achieved, energy requirements and suitable ultimate disposal options for various dewatering methods. The solids concentrations shown in Table 4.11 assume that the sludges have been properly conditioned.

If sludge is to be disposed of in sludge lagoons, dewatering may not be necessary unless it is justifiable for economic reasons relating to haulage costs.

In Alberta, the most prevalent method of sludge disposal is by land application on agricultural lands. Due to the fact that the ammonium nitrogen content of sludges is largely associated with the liquid fraction of sewage sludges and the acceptability of sludges for spreading on agricultural land relates to minimum ratios of nitrogen to heavy metal concentrations, dewatered sludges will generally be less desirable for final spreading on agricultural lands than liquid sludges. To enable sludges to be handled and spread as liquids, the upper limit for solids content will generally be in the order of 12 percent. This would leave only thickened sludges acceptable for spreading on agricultural land by liquid spreading techniques, and sludge dewatering will not be necessary. For this reason, no design guidelines have been included in this document for sludge dewatering facilities. However, should it become necessary to dewater sludges, the proponent should consult with AESRD before proceeding with the design of the facility.

TABLE 4.11
SLUDGE DEWATERING METHODS AND PERFORMANCE WITH VARIOUS SLUDGE TYPES

Dewatering Method	Solids Capture (%)	Solids Concentrations Normally Achieved	Median Energy Required (m^3/dry tonne)	Suitable Ultimate Disposal Methods	
				Landfill	Agricultural Utilization
Filter Press	90 to 95	Raw Primary + WAS- 30 to 50%) Digested Primary + WAS- 35 to 50%) WAS- (25 to 50%)	360	Yes Yes Yes	No No No
Centrifuge (Solid Bowl)	95 to 99	Raw or Digested Primary + WAS - (15 to 25%) WAS - (12 to 15%)	360	Yes Yes	No No
Belt Filter	85 to 95	Raw or Digested Primary + WAS - (14 to 25%) WAS - (10 to 15%)	130	Yes Yes	No No

4.5 Septage Management

Municipal wastewater facility and the independent septage management facility owners accepting hauled wastewater or septage should follow guidelines outlined in this section and may outline additional requirements as they see appropriate to manage their wastewater system.

4.5.1 Treatment and Disposal of Septage

Septage consists of higher concentration of solids and organics, as well it may contain varying levels of oil and grease and inorganic, making it difficult to handle and treat. The management, treatment and disposal of septage are affected by its chemical and physical characteristics. In addition the method of treatment and disposal must also consider the site specific needs to minimize odour issues, address public heath and minimize environmental risks.

Many different factors affect the physical properties of septage including septic tank size, user habits, and pump-out frequency. Therefore a good knowledge of septage characteristics and variability is important in determining appropriate treatment or disposal methods.

An engineering evaluation of the wastewater treatment facility may be required when the volume of the septage received by the wastewater facility exceeds 5% of the wastewater treatment facilities' capacity for instantaneous flow rate or mass loading of solids, metal or other parameter. The engineering evaluation should include process impact analysis through process modeling, mass balances or another approved method.

4.5.2 Septage Management Systems

Septage receiving system including the receiving facility, pipes, holding chamber and pumps should be designed for a minimum of 10 years design flow with provision for expansion to handle 20 or 25-year design flow.

The design of the septage receiving facility at the wastewater treatment plant or at an independent septage management facility should include:

1. Capability to control access to the facility;

2. A hard surface unloading ramp sloped to a drain to collect spillage. The ramp drainage must discharge to the treatment facility and must avoid or limit collection of storm water.

3. Capability to monitor and track septage haulers discharging to the facility;

4. Capability to collect a manifest from each septage hauler;

5. Automated sampling systems to collect a representative sample of truck load discharging at the facility as well as composite sample of loads over a 24-hour period;

6. Odour management system;

7. Flow equalization system;

8. A flexible hose fitted with easy connect coupling to provide for direct connection from the hauling truck outlet to minimize spillage;

9. Heating of discharge chamber and / or receiving piping;

10. Metering and billing systems to monitor septage received and provide accurate billing information to septage haulers and operator(s) of the wastewater treatment facility;

11. Wash down system to clean the septage receiving station;

12. Rock traps, grinding, screening, grit, and / or grease removal where appropriate to protect the downstream treatment units;

13. Valving, piping and short storage for operational flexibility to allow the control of the flow rate and point of septage discharge to the wastewater system; and

14. Provide capability for visual inspections for floating grease, petroleum, oily sheen, visible solids, and colours

In addition, precautionary measures are recommended to control excessive generation of hydrogen sulphide to prevent odours, damage to equipment and hazardous conditions for operators. Chemical or physical treatment including ventilation may be necessary.

4.5.3 Monitoring and Record Keeping for Septage Management

A manifest should be prepared for each hauled load and that this information is collected by the owners of the septage management system. The manifests should include the following information:

1. Name and addresses of location(s) where the septage was collected;
2. Volume and type of load (e.g. holding tank / septic tank) of septage collected from each location;
3. Date septage collected;
4. Name of septage receiving facility discharged to;
5. Date and time when septage was disposed to the receiving facility; and
6. Name and address of septage hauling company.

A septage sampling program should be developed by the owner of the septage management system to obtain representative characteristics of the hauled wastes. Samples should be tested for BOD, COD, TSS, TP, pH, oil and grease, metals, and organics.

The owner of the septage management system prepare summary of the monthly report of the manifests and the sampled collected. A copy of this information should be kept for a minimum period of 5 years. The monthly summary should include the following:

1. Daily septage volume received;
2. Monthly septage volume received from each hauler and total monthly volume;
3. Daily volume of septage diverted to the treatment process (if different from (1.);
4. Results of inspection and random sampling carried out during the month; and
5. Other observations where appropriate.

4.5.4 Criteria for Land Application of Septage

The disposer of septage to land must obtain a Letter of Authorization from Alberta Environment and Sustainable Resource Development prior to land application of septage.

The disposer of septage to land is defined as the person named in a letter of authorization from Alberta Environment and Sustainable Resource Development being authorized to dispose of septage to a land. The disposer shall haul all septic or holding tank waste in a contained vessel and shall only dispose of septic or holding tank waste if the septic tank or holding tank receives only domestic wastewater.

The disposer shall only apply septage to land subject to the following provisions:

1. The disposer shall not apply septage when ice, snow or frozen conditions exist;
2. The disposer shall not apply septage to dry stream and dry intermittent drainage areas; and
3. The disposer shall only apply septage to land that meets the requirements identified in Table 4.12

TABLE 4.12
SITE SUITABILITY

Characteristic	Type of site			
	Casual application site*		Designated application site	
	Acceptable	Unacceptable	Acceptable	Unacceptable
Soil PH	-	-	>6.5	<6.5
Soil Texture	All texture except sand & gravel	Sand & gravel	All texture except sand & gravel	Sand & gravel
Slope of the land	<5%	>5%	<5%	>5%
Depth to groundwater aquifer	-	-	>2m	<2m

* Casual application site is considered as a site where spetage is applied less frequent than once every five years.

4.5.4.1 Separation Distances from Specified Features

The disposer shall apply septage in accordance with the separation distances from specified features set out in Table 4.13**.**

TABLE 4.13
MINIMUM SEPARATION DISTANCES FROM SPECIFIED FEATURES

Feature	Minimum Distance (in metres)	Preferred Distance (in metres)
Rivers, Canals, Creeks, Intermittent Drainage Courses, Lakes, Sloughs, Dugouts	30	50
Water Wells	50	50
Areas Zoned Residential or Devoted to Urban Use	500	800
Occupied dwellings	60	100
Road Allowances	10	20
Public Building Perimeter	10	30
Public Buildings	60	100
School Yard Boundaries	200	500
Cemeteries, Playgrounds, Parks, Campgrounds	200	500
Property Boundary	60	100

4.5.4.2 Methods of Application

The disposer shall only apply septage by injection or surface application.

If surface application is employed, the disposer shall till the land within 48 hours of the surface application to incorporate the stabilized septage with the surface soil material.

4.5.4.3 Application Rates

The disposer shall not exceed the single and annual application rates for casual and designated application sites as specified in Table 4.14.

TABLE 4.14
MAXIMUM APPLICATION RATES FOR CASUAL & DESIGNATED APPLICATION SITES

Waste type	Maximum Single and Annual Application Rates			
	Casual Application Rates*		Designated Application Rates	
	Single application	Annual application	Single application	Annual application
Septic Tank	100 m^3/ha	Not allowed	100 m^3/ha	500 m^3/ha
Holding Tank	100 m^3/ha	Not allowed	100 m^3/ha	300 m^3/ha

* Casual applications rates are rates applied on Casual application site which are considered as sites where septage is applied less frequently than once every five years.

4.5.4.4 Restrictions on Land Use After Application

The disposer shall only apply septage to land intended for the production of forages, oil seeds, small grains, trees and commercial sod.

The disposer shall not apply septage to land that is intended to be used for the production of root crops, vegetable and fruit crops, or dairy farming pasturing within three years of the application of the septage.

4.5.4.5 Requirement for Written Permission from Landowner

The disposer shall obtain written permission from the landowner prior to the application of septage.

4.5.4.6 Record Keeping

The disposer shall maintain a minimum of the following records:

1. Address of all clients;
2. Volume of septage collected from each client;
3. Land location of disposal of the septage collected;
4. Method and application rate of the septage disposed; and
5. Date of application of the septage and date of till if surface application was employed.

All records shall be kept by the disposer for a minimum of five (5) years from the date of the septage application.

4.6 Wastewater Systems Operation and Monitoring

4.6.1 General

The proper operation and maintenance of wastewater systems is essential to produce highest quality of treated effluent and to ensure the protection of public health and the environment. It is therefore important that programs and activities such as good operator training, emergency response planning, corrective action measures, etc. are in place to ensure a reliable and well-operated wastewater system.

4.6.2 Reliability

1. The wastewater system must produce effluent to meet the required limits at all times. Consideration should be given to optimize operation of the system to handle both dry weather and wet weather flows.

2. The owner should ensure that the system is operated, maintained, and has appropriate backup facilities to protect against failures of the power supply, treatment process, equipment, or structure.

4.6.3 Operations

1. The wastewater systems must be managed and operated in accordance with the EPEA approval of the systems. The wastewater treatment facilities must be operated to produce effluent that meet the standards detailed in Tables 3.1 and 3.2.

2. Non-domestic discharges should not interfere with the operation of the treatment plant, nor should it impact on the treatability of the wastewater and affect the performance of the plant.

3. The owner should ensure the development and implementation of an emergency response plan as part of the operations program, for emergencies such as pipeline breakage or accidental spills of any toxins to sewers and / or treatment plant. The plan should include:

    a. General procedures for routine or major emergencies within the wastewater system; and

    b. A contingency plan for facilities becoming inoperable in a major emergency.

4. The plant should be operated within its design capacity.

5. The owner should take preventative or corrective action as directed by AESRD when results of an inspection conducted by AESRD or monthly returns indicate conditions which are currently or may become a detriment to system operations.

4.6.4 Facility Classification and Operator Requirements

4.6.4.1 Facility Classification

On recommendation from Water and Wastewater Operators Certification Advisory Committee, AESRD will classify all wastewater facilities. Facility classification may also be reviewed upon request by the owner or authorized representative. The classification of Wastewater Collection (WWC) system is based upon the population served by the facilities while the classification of Wastewater Treatment (WWT) facilities is based upon a range of points as shown in Tables 4.15(a) and 4.15(b).

Table 4.16 summarizes the classification system. The classification system is based on the "degree of difficulty to operate" a facility. The Alberta system is similar to those used across Canada and the United States.

4.6.4.2 Requirement For Having Certified Operators

In accordance with EPEA, day-to-day operations of wastewater systems must be supervised by one or more persons who hold a valid certificate of qualification for the type of class of facility concerned. The Approval for each facility will state the required number of certified operators and their required level of certification.

4.6.4.3 Responsibility of Operators

It is the responsibility of certified operators to know and understand the terms and conditions in the operating Approval for their facility. It is also their responsibility to understand the certification requirements for operators of their facilities as indicated by the Approval or by the Certification Guidelines.

It is necessary that the chief operator ensure current certification for operators as required by the Approval or by the Certification Guidelines. It is also important that each facility has a contingency plan so that certified operator requirements are met in cases of planned absences (e.g. vacation), unplanned absences (e.g. illness), or change of staff (e.g. retirement).

Certified operators along with the Approval or Registration holder are also responsible to establish or understand contingency plans for each facility that ensure that the Approval requirements, with respect to certified operators, are met at all times. This is important during normal operation or in the cases of planned absences (e.g. vacation), unplanned absences (e.g. illness), or change of staff (e.g. retirement).

4.6.4.4 Responsibility of Facility Owners

It is the legal responsibility of the owner of each facility to be aware of the requirements of the Approval and to ensure that the requirements are met. The Approval issued by AESRD will designate the minimum number and level of certification of key operations personnel. It is important that facility owners develop an internal program so that substitute or replacement personnel are available when necessary.

TABLE 4.15(a)
CLASSIFICATION OF WASTEWATER TREATMENT PLANTS (WWT)

ITEM	POINTS
Size	
Maximum population equivalent (P.E.) served, peak day (1 pt/10, 000 P.E. or part) (Population Served____________)	(0 – 5)
Design flow (avg. day) or peak month's flow (avg. day), whichever is larger (1 pt/5000 m^3/day	(0 – 5)
Effluent Discharge	
Receiving stream (sensitivity)*	(0 - 6)*
Land disposal – evaporation	(2)
Subsurface disposal	(4)
Variation in Raw Wastewater (slight to extreme)*	(0 - 6)*
Pretreatment	
Plant pumping of main flow	(3)
Screening, comminution	(3)
Grit Removal	(3)
Chemical pre-treatment except chlorination, enzymes	(4)
Primary Treatment	
Primary clarifiers	(5)
Combined Sedimentation / digestion	(5)
Secondary Treatment	
Trickling filter w / sec.clarifiers or RBC	(10)
Activated sludge w / sec. clarifiers (including ext. aeration and oxidation ditches)	(15)
Stabilization ponds without aeration	(5)
Aerated	(8)
Dissolved air flotation (not sludge)	(8)
Advanced Waste Treatment	
Polishing pond	(2)
Chemical / physical – without secondary	(15)
Chemical / physical – following secondary	(10)
Biological or chemical / biological	(12)
Ion exchange	(10)
Reverse Osmosis, electro dialysis, membranes, cloth filters	(15)
Chemical recovery, carbon regeneration	(4)
Solids Handling	
Conditioning	(2)
Thickening	(5)
Anaerobic digestion	(10)
Aerobic digestion	(6)
Evaporation sludge drying	(2)
Mechanical dewatering	(8)
Solids reduction (incineration, wet oxidation)	(12)
Composting (on site)	(8)

Table 4.15(a) continued

Disinfection Chlorination or comparable On-site generation	 (5) (5)
Use of SCADA (see note)	(0 – 6)*
Laboratory Control by Plant Personnel Bacteriological (complexity)* **Chemical / Physical (complexity)***	 (0 – 10)* (0 – 10)*

* See Table 4.15(b)

TABLE 4.15(b)
WASTEWATER TREATMENT PLANT CLASSIFICATION POINT GUIDE

Note: Each unit process should have points assigned only once

ITEM	POINTS
EFFLUENT DISCHARGE Receiving Stream Sensitivity The Key concept is the degree of dilution provided under low flow conditions Suggested point values are:	**0 - 6**
No discharge	0
Secondary treatment is adequate	1
More than secondary treatment is required	2
Stream conditions are very critical (run dry, for example)	3
Effluent used in a direct recycle and reuse system	6

VARIATION IN RAW WASTEWATER (slight to extreme) The key concept is frequency and / or intensity of deviation or excessive variation from normal or typical fluctuations; such deviation can be in terms of strength, toxicity, shock loads etc. Suggested point values are:	**0 - 6**
Variations do not exceed those normally or typically expected	0
Recurring deviations or excessive variations of 100 to 200 percent in strength and / or flow	2
Recurring deviations or excessive variations of more than 200 percent in strength and / or flow	4
Raw wastes subject to toxic waste discharges	6

LABORATORY CONTROL BY PLANT PERSONNEL Bacteriological / biological (complexity) The Key concept is to credit bacti / bio lab work done on-site by plant personnel	**0 - 10**
Lab work done outside the plant	0
Membrane filter procedures	3
Use of fermentation tubes or any dilution methods; fecal coliform determination	5
Biological identification	7
Virus / parasite studies or similarly complex work conducted on-site	10

Table 4.15(b) continued

Chemical / physical (complexity)	**0 – 10**
The key concept is to credit chemical / physical lab work done on-site by plant personnel	
Lab work done outside the plant	**0**
Push button or visual methods for simple test such as pH, settleable solids	**Up to 3**
Additional procedures such as DO, COD, BOD, solids, gas analysis, titrations, volatile content	**Up to 5**
More advanced determinations such as specific constituents; nutrients, total oils, phenols, etc.	**Up to 7**
Highly sophisticated instrumentation such as atomic absorption and gas chromatography	**10**

USE OF SCADA	**0 - 6**
Provide data with no process adjustment	0
Provide data with limited process adjustment	2
Provide data with moderate process adjustment	4
Provide data with extensive or total process adjustment	6

TABLE 4.16
FACILITY CLASSIFICATION SYSTEM

FACILITY	BASED UPON	I	II	III	IV
WWC*	Population Served	1500 or fewer	1501-15,000	15,001-50,000	50,001 or more
WWT	Range of Points (Table 4.12(a))	30 or fewer	31-55	56-75	76 or more

Notes: AESRD may adjust the classification of a facility if the point system does not reflect the actual complexity of that facility.
WWC - Wastewater Collection
WWT - Wastewater Treatment
Wastewater pumping and transmission may be part of either wastewater treatment or wastewater collection but, alone, it is not considered to be either wastewater treatment or wastewater collection.
*Simple "in-line" treatment (such as odour control) is considered an integral part of the collection system.

4.6.4.5 Facilities Staffing Requirements: Certified Operators

For **Class I Wastewater Treatment, and Wastewater Collection facilities**, there **must** be a certified Level I (or higher) operator in charge of the day–to–day operation of that facility. The exception to this is when the approval or registration states that an operator with a Small Systems certificate is acceptable. A back–up certified operator is **recommended**.

For **Class II Wastewater Treatment and Wastewater Collection facilities**, there **must** be a certified Level II (or higher) operator in charge of the day–to–day operation of that facility. It is **recommended** that an assistant operator with Level I or II certification be available.

For **Class III Wastewater Treatment facilities serving a population under 1,500**, there **must** be a certified Level III (or higher) operator in charge of the day–to–day operation. It is further **recommended** that other operators be certified.

For Class III Wastewater Treatment serving a population of 1,501 – 15,000, there **must** be a certified Level III (or higher) operator in charge of the day–to–day operation. There **must** also be at least one other operator certified at Level I or higher. It is further **recommended** that other operators be certified.

For Class III Wastewater Treatment facilities serving a population of 15,001 – 50,000, there **must** be a certified Level III (or higher) operator in charge of the day–to–day operation. There **must** also be at least one other operator certified at Level II or higher. It is **recommended** that other operators be certified.

For Class III Wastewater Collection systems serving a population of 15,001 – 50,000, there **must** be a certified Level III (or higher) operator in charge of the day–to–day operation. There **must** also be at least one other operator certified at Level II or higher. It is **recommended** that other operators be certified.

For Class III Wastewater Treatment facilities serving a population over 50,000, there **must** be a certified Level III (or higher) operator in charge of the day–to–day operation. There **must** also be another operator certified at Level II or higher to act in the absence of the charge operator and at least one other operator certified at Level I or higher. There **must** be at ***least*** one certified operator (any level) for each shift when shift operation is required. It is **recommended** that other operators be certified.

For Class IV Wastewater Treatment facilities serving a population up to 200,000, there **must** be a Level IV operator in charge of the day–to–day operation. There **must** also be two Level III (or higher) operators to act in the absence of the Level IV operator. There **must** be at least one Level II or higher certified operator for each shift when shift operation is required. It is **recommended** that other operators be certified.

For Class IV Wastewater Collection systems serving a population of 50,001 or more, there **must** be a Level IV operator in charge of the day–to–day operation. There **must** also be two Level III (or higher) operators to act in the absence of the Level IV operator. There **must** be at least one Level II or higher certified operator for each shift when shift operation is required. It is **recommended** that other operators be certified.

For Class IV Wastewater Treatment facilities serving a population over 200,000, there **must** be a certified Level IV operator in charge of the day–to–day operation. There **must** be at least one other certified Level IV operator to act in the absence of the charge operator. There **must** be a third operator who is certified at either Level III or IV and there **must** be at least one Level II (or higher) certified operator for each shift when shift operation is required. It is **recommended** that other operators be certified.

4.7 System Monitoring

4.7.1 General

Alberta Environment and Sustainable Resource Development considers monitoring to fall into one of the following categories:

1. operational monitoring
2. treatment performance and compliance monitoring
3. issue oriented and follow-up monitoring.

Types of monitoring are discussed in detail in the next few sections.

4.7.1.1 Sampling Procedures and Analytical Methods

1. The usefulness of any monitoring program depends to a large extent on the sampling procedures used. It is important to ensure that the sample collected is truly representative of the wastewater stream.

2. Based on the type of analysis, AESRD would require two types of samples to be collected, "composite" or "grab."

   "Composite sample" means a sample consisting of not less that twenty-four discrete portions of equal volume collected as follows:

   a. at time intervals directly proportional to the flow rate of the liquid being sampled during each time interval, with a minimum of one discrete sample collected every hour over a period of 24 hours, or

   b. sequentially at regular time intervals over a period of 24 hours.

   "Grab sample" means an individual sample collected in less than 15 minutes and which is representative of the wastewater being sampled.

3. The owner should ensure that:

   a. Collection and preservation of samples and all analytical procedures are:

      - in accordance with the latest edition of "Standard Methods for the Examination of Water and Wastewater," as published by the American Public Health Association, American Water Works Association, and the Water Pollution Control Federation; or

      - by a method outlined in the most recent edition of the "Methods Manual for Chemical Analysis of Water and Wastes" or "Methods Manual for Chemical Analysis of Trace Organics and Pesticides in Environmental Samples," published by Alberta Environmental Protection; or

      - by an alternative method approved by AESRD; or

   b. Collection, preservation and the analysis of samples are performed by a laboratory approved by AESRD.

4.7.1.2 Approval of Analytical Procedures

Owners should ensure that laboratories obtain approval from AESRD for the use of any analytical procedures not included in the "Standard methods."

The laboratory would be required to follow a protocol established by AESRD for the approval of analytical procedures not included in the "Standard Methods."

4.7.2 Operational Monitoring

The extent and the complexity of operational monitoring is dependent on the size and the type of the facility. For instance, there are a number of operating variables which are vitally important to the proper functioning of an activated sludge system. Some of these are under the Operator's control, and some are not. Operational monitoring requirements are established both for specific process control purposes, and to ensure that a facility receives good operational attention on a regular basis.

4.7.2.1 Activated Sludge Systems

Table 4.17 lists some of the significant parameters that should be monitored to ensure proper operation of an activated sludge system.

TABLE 4.17
OPERATIONAL MONITORING

Parameters	Point of Measurement	Requirement	Type of Sample	Monitoring Frequency
Flow (raw)	Headworks	Not to exceed design capacity	N/A	Daily from totalizer
Flow (treated)	Prior to Outfall	N/A	N/A	Daily from totalizer
Peak flow (raw)	Headworks	N/A	N/A	Daily from strip - chart or PLC
TSS (raw)	Headworks	N/A	Composite	Daily
TSS (primary)	Downstream of primary clarifier	Varies; required to determine the aeration capacity	Composite	Daily
TSS (RAS)	RAS line	Varies; required to control solids in aeration tank	Grab/Composite	Daily
TSS (treated)	Prior to outfall	Refer to compliance limit	Composite	Daily
MLSS	Aeration tank	800-2000 mg/L (without nitrification)	Grab	Daily
		2000-4000 mg/L (with nitrification)	Grab	Daily
CBOD (raw)	Headworks	N/A	Composite	Daily
COD (primary)	Downstream of primary clarifier	Varies; required to determine the aeration capacity	Composite	Daily
CBOD (treated)	Prior to outfall	Refer to compliance limit	Composite	Daily
Sludge Volume Index	Aeration tank/calculated	< 150 mL/g	Grab	Daily
Sludge Age	Calculated	3-10 days (without nitrification)	N/A	Daily
		10-15 days (with nitrification)	N/A	Daily
F:M ratio	Calculated	0.05 - 0.5	N/A	Twice Weekly
Dissolved Oxygen	Aeration tank	2 mg/L	N/A	Continuous

4.7.2.2 Wastewater Lagoons

Table 4.18 outlines the operational monitoring requirements for wastewater stabilization ponds.

TABLE 4.18
OPERATIONAL MONITORING

Requirements	Monitoring Frequency
Actual dates and duration of discharge	Annually for every discharge
Volume of discharge	Annually for every discharge
Inspection of the discharge route	Prior each discharge the discharge route should be inspected to identify changes / site specific concerns and modify discharge practice as necessary. Issues such as stream flows, other discharges either upstream or down stream, stream blockage, etc… should be documented and addressed as necessary.
Monitoring of each groundwater observation well for: water level, TKN NH_3-N, NO_3-N, NO_2-N, TDS, COD, and any other parameter as determined by AESRD.	For new lagoons, four times in each quarter of the first year of operation. The first analysis from each well prior to putting the new lagoon into operation. The following three analyses approximately three months apart to cover all seasons of the year.
Monitoring of each groundwater observation well for water level during the discharge period.	One set of readings immediately before discharge, one set immediately after discharge, and one set approximately one month after the end of the discharge period.

4.7.3 Treatment Performance and Compliance Monitoring

Treatment performance and compliance monitoring will be dependent on a number of factors, including:

- type of treatment (mechanical secondary treatment plants, mechanical tertiary treatment plants, aerated lagoons, wastewater stabilization ponds)
- type of discharge (continuous, intermittent)
- type of receiving body (water or land).

The Wastewater and Storm Drainage Regulation and the Wastewater and Storm Drainage (Ministerial) Regulation (119/93 and 120/93) require physical, microbiological, radiological or chemical analyses of wastewater and storm drainage samples for those parameters specified by AESRD. This section outlines the specific parameters that have to be monitored, including the sample location and the monitoring frequency, for different types of treatment method, discharge and receiving body.

4.7.3.1 Secondary Treatment - Mechanical (for current population < 20,000)

1. Continuous Discharge to a body of water

TABLE 4.19
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to outfall	Monthly averaging of daily samples shall not exceed 25 mg/L	Composite	Daily
TSS	Prior to outfall	Monthly average of daily samples shall not exceed 25 mg/L	Composite	Daily

2. Intermittent Discharge to a Body of Water

TABLE 4.20
TREATMENT PERFORMANCE AT COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to storage cell	Monthly average of three times per week; samples shall not exceed 25 mg/L	Composite	Three times per week during the period of discharge to storage cell
CBOD	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period **should not** exceed three weeks.
TSS	Prior to storage cell	Monthly average of five times per week samples shall not exceed 25 mg/L	Composite	Five times per week during the period of discharge to storage cells, excluding statutory holidays.
TSS	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.

3. Continuous and / or intermittent discharge to land (Effluent Irrigation)

TABLE 4.21
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to irrigation	< 100 mg/L	Grab/composite*	Twice annually, prior to and on completion of a major application event
TSS	Prior to irrigation	< 100 mg/L	Grab/composite*	Twice annually, prior to and on completion of a major application event
EC	Prior to irrigation	< 2.5 mmhos/cm	Grab/composite*	Twice annually, prior to and on completion of a major application event
SAR	Prior to irrigation	< 9	Grab/composite*	Twice annually, prior to and on completion of a major application event
pH	Prior to irrigation	6.5 - 9.5	Grab/composite*	Twice annually, prior to and on completion of a major application event
Total Coliform	Prior to irrigation (golf course/parks only)	Geometric mean of weekly samples (if storage provided) and daily samples (if storage not provided), in a 30-day period shall not exceed 1000/100 mL	Grab	Daily**/weekly during the irrigation season
Fecal Coliform	Prior to irrigation (golf course/parks only)	Geometric mean of weekly samples (if storage provided) and daily samples (if storage not provided), in a 30-day period shall not exceed 200/100 mL	Grab	Daily**/weekly during the irrigation season

* Grab sample if storage provided; composite sample if storage not provided

** Frequency of sampling will be reduced if it can be demonstrated with some certainty that bacteriological quality of effluent is consistent and the probability of variance is minimal.

4.7.3.2 Tertiary Treatment - Mechanical (for current population > 20,000)

1. Continuous discharge to a body of water

TABLE 4.22
TREATMENT PERFORMANCE AND COMPLIANCE

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to outfall	Monthly average of daily samples shall not exceed 20 mg/L	Composite	Daily
TSS	Prior to outfall	Monthly average of daily samples shall not exceed 20 mg/L	Composite	Daily
TP	Prior to outfall	Monthly average of daily samples shall not exceed 1 mg/L	Composite	Daily
NH_3-N	Prior to outfall	Assessed on a site specific basis	Composite	Assessed on a site specific basis
Total Coliform	Prior to outfall	Geometric mean of daily samples in a 30 day period shall not exceed 1000/100 mL	Grab	*Daily
Fecal Coliform	Prior to outfall	Geometric mean of daily samples in a 30 day period shall not exceed 200/100 mL	Grab	*Daily

* Frequency of sampling will be reduced if it can be demonstrated with some certainty that bacteriological quality of effluent is consistent and the probability of variance is minimal.

2. Intermittent Discharge to a Body of Water

TABLE 4.23
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to storage cell	Monthly average of three times per week samples shall not exceed 20 mg/L	Composite	Three times per week during the period of discharge to storage cell
CBOD	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period shall not exceed three weeks, unless local conditions preclude this rate of discharge.
TSS	Prior to storage cell	Monthly average of five times per week samples shall not exceed 20 mg/L	Composite	Five times per week during the period of discharge to storage cells, excluding statutory holidays.
TSS	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks unless local conditions preclude this rate of discharge.
TP	Prior to storage cell	Monthly average of five times per week samples shall not exceed 1 mg/L	Composite	Five times per week during the period of discharge to storage cells, excluding statutory holidays

Table 4.23 - Continued

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
TP	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.
NH_3-N	Prior to outfall	Assessed on a site specific basis	Grab	Assessed on a site-specific basis.
Total Coliform	Prior to storage cell	Geometric mean of three times per week; samples in a calendar month shall not exceed 1000/100 mL	Grab	*Three times per week during the period of discharge to storage cell.
Total Coliform	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.
Fecal Coliform	Prior to storage cell	Geometric mean of three times per week samples in a calendar month shall not exceed 200/100 mL	Grab	*Three times per week during the period of discharge to storage cell.
Fecal Coliform	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.

* Frequency of sampling will be reduced if it can be demonstrated with some certainty that bacteriological quality is consistent and the probability of variance is minimal.

3. Continuous and / or intermittent discharge to land (Effluent Irrigation)

   Tertiary (mechanical) treatment performance and compliance monitoring for effluent irrigation is the same as for secondary (mechanical) treatment monitoring for effluent irrigation. Please refer to Table 4.21.

4.7.3.3 Aerated Lagoons (for current population < 20,000)

1. Continuous discharge to a body of water

TABLE 4.24
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to outfall	Monthly averaging of weekly samples shall not exceed 25 mg/L	Grab	Weekly

2. Intermittent discharges to a body of water

TABLE 4.25
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to storage cell	Monthly average of weekly samples shall not exceed 25 mg/L	Grab	Weekly during the period of discharge to storage cell
CBOD	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.

3. Continuous and / or intermittent discharge to land (Effluent Irrigation)

   Aerated lagoon performance and compliance monitoring for effluent irrigation is the same as for secondary (mechanical) treatment monitoring for effluent irrigation. Please refer to Table 4.21.

4.7.3.4 Aerated Lagoons (for current population > 20,000)

1. Continuous discharge to a body of water

TABLE 4.26
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to outfall	Monthly average of weekly samples shall not exceed 20 mg/L	Grab	Weekly
TP	Prior to outfall	Monthly average of weekly samples shall not exceed 1 mg/L	Grab	Weekly
NH_3-N	Prior to outfall	Assessed on site specific basis	Grab	Assessed on site-specific basis.
Total Coliform	Prior to outfall	Geometric mean of weekly samples in a calendar month shall not exceed 1000/100 mL	Grab	Weekly
Fecal Coliform	Prior to outfall	Geometric mean of weekly samples in a calendar month shall not exceed 200/100 mL	Grab	Weekly

2. Intermittent discharge to a body of water

TABLE 4.27
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Parameter	Point of Measurement	Requirement	Type of Sample	Minimum Monitoring Frequency
CBOD	Prior to storage cell	Monthly average of weekly samples shall not exceed 20 mg/L	Grab	Weekly during the period of discharge to storage cell
CBOD	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.
TP	Prior to storage cell	Monthly average of weekly samples shall not exceed 1 mg/L	Grab	Weekly during the period of discharge to storage cell.
TP	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.
NH_3-N	Prior to outfall	Assessed on a site specific basis	Grab	Assessed on a site specific basis
Total Coliform	Prior to storage cell	Geometric mean of weekly samples in a calendar month shall not exceed 1000/100 ml	Grab	Weekly during the period of discharge to storage cell
Total Coliform	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.
Fecal Coliform	Prior to storage cell	Geometric mean of weekly samples in a calendar month shall not exceed 200/100 ml	Grab	Weekly during the period of discharge to storage cell.
Fecal Coliform	Prior to outfall	None	Grab	Once, on the second day of the discharge period. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.

4.7.3.5 Wastewater Lagoons

TABLE 4.28
TREATMENT PERFORMANCE AND COMPLIANCE MONITORING

Type	Minimum Requirements
Wastewater stabilization pond built to the specified design configuration as per Section 3.4.1.2.	Effluent quality standard not specified Discharge from the storage cell once a year between late spring and fall. Early spring discharged allowed under exceptional circumstances Prior each discharge the discharge route should be inspected to identify changes / site specific concerns and modify discharge practice as necessary. Issues such as stream flows, other discharges either upstream or down stream, stream blockage, etc… should be documented and addressed as necessary. Discharge period should not exceed three weeks, unless local conditions preclude this rate of discharge.

4.7.4 Issue Oriented and Follow-up Monitoring

1. Follow-up action by the owner may be required when the system does not meet or produce effluent to meet the standards stipulated in Section 3.0: Wastewater Systems – Performance and Design Standards. Issue oriented monitoring and follow-up actions for various incidents are outlined in Table 4.29.

2. When a violation of the prescribed effluent standard occurs, the owner should:

   a. notify AESRD in accordance with Section 4.9;

   b. determine the cause of the problem; and

   c. take action as directed by AESRD.

4.7.4.1 Activated Sludge System

Issue oriented monitoring and follow-up actions for various incidents in Activated Sludge Systems are outlined in Table 4.29.

TABLE 4.29
ISSUE ORIENTED AND FOLLOW-UP MONITORING

Incident	Parameter	Point of Measurement	Type of Sample	Monitoring Frequency	Follow-Up
Plant by-pass	flow	By-pass line	N/A	Each incident	-
	CBOD		Grab/composite		
	TSS		Grab/composite		
	Coliform (total)		Grab		
	Coliform (fecal)		Grab		
	NH_3		Grab/composite		
	Total.P		Grab/composite		
Sludge bulking	CB0D (primary)	Downstream of primary clarifier	Grab/composite	For the duration of event	Adjust sludge age, adjust D.O levels, Adjust WAS rates, chlorination of RAS
	TSS (primary)	Downstream of primary clarifier	Grab/composite		
	pH (primary))	Downstream of primary clarifier	Grab/composite		
	MLSS	aeration tank	Grab		
	D.O.	aeration tank	N/A		
	NO_3-N	RAS line	Grab		
Rising Sludge/ Denitrification	CBOD (primary)	Downstream of primary clarifier	Grab/composite	For the duration of the event	Increase F/M ratio, Adjust RAS, Adjust D.O. levels, Step-feed influent
	TSS (primary)	Downstream of primary clarifier	Grab/composite		
	MLSS	aeration tank	Grab		
	NO_3-N	RAS line	Grab		
Primary Sludge Septicity	Flow Retention time Density CBOD (raw) TSS (raw)	Primary sludge line	N/A	For the duration of the event	Increase sludge wasting, monitor retention time

4.8 Record Keeping

All records should bear the signature of the operator in responsible charge of the wastewater system or his or her representative. Owners shall keep these records available for inspection by AESRD and shall send the records to AESRD if requested.

The owners shall keep the following records and effluent quality analyses:

1. All daily records for treatment performance and compliance monitoring for five years.

2. All daily records for operational monitoring for three years.

3. Actual laboratory reports may be kept or data may be transferred to tabular summaries, provided the following information is included:

    a. the date, place and time of sampling, and the name of the person collecting the sample;

    b. identification of the sample type (compliance sample, operational sample, special purpose sample);

    c. date of analysis;

    d. laboratory and person responsible for performing analysis;

    e. the analytical method used; and

    f. the results of the analysis.

4. Records of action taken by the system to correct violations of effluent standards or operating approval requirements.

5. Copies of project reports, construction documents and related drawings, inspection reports and approvals should be kept for the life of the facility.

4.9 Reporting

1. Reporting requirements shall be in accordance with the operating approval for the facility, issued by AESRD.

2. The owner shall report to AESRD immediately up on finding:

    a. the violation of prescribed effluent standards for that facility; and

    b. the failure to comply with the treatment performance and compliance monitoring requirements.

3. Immediate notification by telephone to 1-800-222-6514 shall be made, followed by a written report to AESRD within one week and remedial action carried out as per EPEA Division 1, Sections 110, 111 and 112 in the event of:

- any discharge of raw or partially treated wastewater
- any unauthorized discharge or accidental spill of raw or partially treated wastewater
- any overflow from the wastewater collection or treatment system.

4. At least one week prior to draining of wastewater stabilization ponds, the owner shall notify AESRD in writing of the proposed discharging schedule.

5. The owner should compile and submit an annual report on or before February 28 of the following year on which the information was collected. The report should include the following:

    a. a monthly summary of all operational and compliance monitoring for that particular facility, as identified by AESRD;

    b. a summary of approval contraventions and remedial measures taken; and

    c. a summary of any permanent upgrading works undertaken during the year.

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