Section 5 – Sewerage Works

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5. Sewerage Works

The Engineer shall follow these guidelines and comply with all requirements of the DOEC in the design of any sewerage works project. In addition, the Engineer should confer with the DOEC prior to undertaking the design of any major infrastructure project.

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

5.1. Regulations

Reference should be made to *The Environmental Control Water & Sewage Regulations, 2003* regarding the allowable discharges to a sanitary sewer and receiving environment.

5.2. Sewers and Appurtenances

Sanitary sewers shall be designed of suitable materials, be of sufficient capacity, and be installed at required grades and alignment to carry sewage from residential, commercial, institutional and industrial establishments to the treatment plant or point of disposal. Infiltration of groundwater shall be excluded as far as possible and connection of stormwater sources such as roof, yard, street and foundation drains is prohibited.

5.2.1. Types of Sewers

Sewers shall be classified broadly with respect to use as follows:

Sanitary sewers carry domestic sewage from houses, business, buildings and other public and private establishments. They may be designed to remove certain industrial wastes, but clean cooling waters should not be discharged to sanitary sewers.

Storm sewers carry storm water and surface drainage, street wash and other wash waters or drainage, but exclude sanitary sewage and industrial wastes and effluent from septic tanks or other treatment processes.

Combined sewers receive both sanitary sewage and storm runoff. To the greatest extent possible, these shall be prohibited.

Collector (main) sewers are those sewers to which one or more branch sewers are tributary and which serve as inlets to sewerage works system.

Intercepting sewers receive dry weather flow from a number of transverse sewers or outlets and frequently additional predetermined quantities of storm water (if from a combined system) and carry such water to a point for treatment or disposal.

Outfall sewers receive sewage from a collecting system and/or a sewerage treatment plant and carry it to a point of final discharge.

Criteria for the design of all the above-mentioned sewers, except storm and outfall sewers, are generally the same. Certain features, which are not common to the design of all systems, have been included in this section. Additional design criteria for outfall sewers are discussed in Section 5.2.15.

5.2.2. Capacity of Sewers

The flow rates of sewage or wastewater for which the sewer capacity should be provided shall be determined from careful consideration of the present and future quantities of domestic sewage, commercial and industrial wastes, groundwater infiltration and any other unavoidable contributions. The design of extensions to existing sanitary sewer systems shall be based on experience if adequate records are not available, or when new systems are being established, the design criteria shall be substantiated by data for similar systems. In determining the required capacities of sanitary sewers, the following factors should be considered:

1. Peak hourly rates of flow;

2. Peak rates of flow from industrial plants; and

3. Groundwater infiltration.

5.2.3. Sewage Flows

Sewage flows are made up of waste discharges from residential, commercial, institutional and industrial establishments, as well as extraneous non-waste flow contributions such as groundwater and surface runoff.

5.2.3.1. Extraneous Sewage Flows

5.2.3.1.1. Inflow

When designing sanitary sewer systems, allowances must be made for the leakage of groundwater into the sewers and building sewer connections (infiltration) and for other extraneous water entering the sewers from such sources as leakage through manhole covers, foundation drains, roof down spouts, etc. Due to the extremely high peak flows that can result from roof down spouts, they should not, in any circumstances, be connected directly, or indirectly via foundation drains, to sanitary sewers. Studies have shown that flows from this source can result in gross overloading of sewers, pumping stations and sewage treatment plants for extended periods of time. The DOEC recommends that foundation drainage be directed either to the surface of the ground or into a storm sewer system, if one exists.

5.2.3.1.2. Infiltration

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

5.2.3.2. Design Criteria

5.2.3.2.1. Development with Separate Storm and Sanitary Sewer Systems

Developments using approved piping materials with separate storm and sanitary sewer systems meeting infiltration allowances specified by Department of Municipal and Provincial Affairs *Municipal Water, Sewer and Road Specifications* shall apply a peaking factor to the average daily flow only. There shall be no need to allow for extraneous flows.

5.2.3.2.2. Development with Combined Sewer Systems – Estimating Wastewater Flows

The following sections outline methodologies for quantifying wastewater flows. From both quantitative and a qualitative point of view, owners of wastewater systems are encouraged to develop and implement policies and programs to promote "at source reduction" for any and all contaminants in wastewater.

5.2.3.2.2.1. Residential (Population-Generated)

Every effort should be made to establish design flows using measured data (see Section 5.2.2). As a second preferred option, measured flow data from similar applications may be used. If no flow data exists for the system or data for similar systems exist, the peak (population-generated) flow for a residential population may be determined by the following formula:

$$Q_{pdw} = \frac{G \times P \times Pf}{86.4}$$

Where: Q_{pdw} = peak dry weather design flow rate (L/s);
G = per capita average daily design flow (L/d), for residential use 340 L/capita/day;
P = the design contributing population in thousands; and
Pf = a "peaking factor".

The peaking factor (Pf) should be the larger of 2.5 or Harmon's Peaking Factor where:

$$Harmon's\ Peaking\ Factor = 1 + \frac{14}{4 + P^{0.5}}$$

Where: P = the design contributing population in thousands.

5.2.3.2.2.2. Commercial/Institutional and Industrial

For detailed system design, the average wastewater flows from commercial/ institutional and industrial land use areas are to be estimated as set out in Table 5.1 or by actual documented usage.

Table 5.1
Estimated Sewage Flows

Type of Establishment		**Unit**	**L/day**
Residential	Private Dwelling	Person	340
	Apartment Buildings	Person	340
Transient Dwelling Units	Hotels	Bedroom	340
	Lodging Houses and Tourist Homes	Bedroom	270
	Motels and Tourist Cabins	Bedroom	270
Camps	Trailer Camps (Private Bath)	Person	340
	Trailer Camp (Central Bath, etc)	Person	230
	Trailer Camp (Central Bath, Laundry)	Person	300
	Luxury Camps (Private Bath)	Person	340
	Children's Camps (Central Bath, etc)	Person	230
	Labour Camps	Person	180
	Day Camps-No Meals	Person	70
Restaurants (Incl.Washrooms)	Average Type (2 × Fire Commissioner's capacity)	Patron	70
	Bar/Cocktail Lounge (2 × Fire Commissioner's capacity)	Patron	25
	Short order or Drive-In Service	Patron	25
Clubhouses	Residential Type	Person	340
	Non-Residential (Serving Meals)	Person	160
Institutions	Hospitals	Bed	900
	Other Institutions	Bed	375
Schools	Elementary (No Shower or Cafeteria)	Person	50
	With Cafeteria	Person	70
	With Cafeteria and Showers	Person	90
	With Cafeteria, Showers and Laboratories	Person	115
	Boarding	Person	340
Theatres	Theatre (Indoor)	Seat	25
	Theatre (Drive-In With Food Stand)	Car	25
Automobile Service Stations	No Car Washing	Car Served	23
	Car Washing	Car washed	340
Miscellaneous	Stores, Shopping Centres & Office Buildings	M2	6
	Factories (8-hour shift)	Person	115
	Self-service Laundries	Wash	230
	Bowling Alleys	Alleys	900
	Swimming Pools and Beaches	Person	70
	Picnic Parks (With Flush Toilets)	Person	50
	Fairgrounds (based upon average attendance)	Person	25
	Assembly Halls	Seat	25
	Airports (Based on Passenger Use)	Passenger	15
	Churches	Seat	15
	Beauty Parlours	Seat	200
	Barber Shops	Seat	75
	Hockey Rinks	Seat	15

5.2.3.2.2.3. Peak Factor

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

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

5.2.3.2.2.4. Industrial Sewage Flows

Industrial sewage flows will be decided in consultation with the Pollution Prevention Division, of the DOEC.

5.2.3.2.2.5. Flow Variation

Peak sewage flow rates from industrial areas vary greatly depending on such factors as the extent of the area, the types of industries present, the provision of in-plant treatment or regulation of flows, and the presence of cooling waters in the sanitary sewer system.

5.2.3.2.2.6. Flow Rate

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

5.2.3.2.2.7. Average Flow Generation Estimates for Planning

For system planning purposes, when specific land uses and zoning are unknown and the requirements of Section 5.2.3.2.2.2 cannot be defined, the recommended lower limits for estimation of average flow generation (to be used for preliminary planning unless the use of other values is justified with more specific or reliable information) are as follows:

1. Commercial and Institutional Land Uses - The lower limit for Average Flow Generation should be 40 m^3/day/ha (0.46 L/s/ha);

2. Industrial Land Uses - The lower limit for average flow generation should be 30 m^3/day/ha (0.35 L/s/ha);

3. Determination of Peak Dry Weather Flow Rate - Peak dry weather flow rates for specific design areas are to be determined by application of a peaking factor (Pf), related to the average flow rate (Q_{AVG} in L/s) in accordance with the following expression to a maximum value of 5.0:

$$Pf = 6.659\left(Q_{AVG} - 0.168\right)$$

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

$$Q_{PDW} = Pf \times Q_{AVG} = 6.659\left(Q_{AVG} \times 0.832\right)$$

4. Special Considerations - High-Water-Consumption Land Uses - The foregoing guidelines may not be applied to high water consumption land uses such as heavy industry, meat packing plants, breweries, etc. Detailed analysis of the design requirements specific to each development proposal is required in such cases; and

5. Residential Components of Commercial Developments - Where proposed commercial developments include discretionary residential components, the sanitary flow generation from the residential component should be determined in accordance with Section 5.2.3.2.2.1, and is to be included in the determination of the total generation for the development.

5.2.3.2.2.8. Extraneous Flow Allowance - All Land Uses

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

1. General Inflow/Infiltration Allowance - A general allowance of 0.28 L/s/ha should be applied, irrespective of land use classification, to account for wet-weather inflow to manholes not located in street sags and for infiltration flow into pipes and manholes. In addition, a separate allowance for inflow to manholes located in street sags should be added as per the next section.

2. Inflow Allowance - Manholes in Sag Locations - When sanitary sewer manholes are located within roadway sags or other low areas, and are thus subject to inundation during major rainfall events, the sanitary design peak flow rate should be increased by 0.4 L/s for each such manhole, which is applicable for manholes which have been waterproofed. For new construction, all sanitary manholes in sag locations are to be waterproofed. For planning purposes and downstream system design, where specific requirements for an area are unknown, the designer should make a conservative estimate of the number of such manholes, which may be installed in the contributing area based on the nature of the anticipated development, and include an appropriate allowance in the design.

3. Others - In areas where weeping tiles are connected to the sanitary sewer system, an additional amount, based on on-site measurements, should be included in the design flow. The designer should also take into account the pipe material and soil type in determining the extraneous flow allowance.

5.2.3.2.2.9. Total Peak Design Flow Rates
The total peak design flow rates should be the sum of the peak dry weather flow rates as generated by population and land use, and the rate of all extraneous flow allowances, as determined for the design contributing area.

5.2.4. Sewer Size

In no instance shall any street sewer be less than 200 mm diameter.

5.2.5. Sewer Grade

5.2.5.1. Hydraulic Design

It is recommended that sanitary sewers be designed using Manning's formula, with a roughness coefficient *n* of no lower than 0.013 for all smooth-walled pipe materials. Use of lower *n* values may be permitted if deemed justifiable on the basis of research or field data presented. The Manning formula, which is the most commonly used formula for calculating sewer capacity, is as follows:

$$Q = \frac{7.8546 \times 10^{-6}}{n} D^2 R^{2/3} S^{1/2}$$

Where: Q = Flow Capacity of sewer (L/s);
D = Inside diameter of pipe (mm);
R = Hydraulic radius of pipe (mm);
S = Sewer slope; and
n = Roughness factor.

5.2.5.2. Minimum and Maximum Velocities

All sewers shall normally be designed and constructed to give mean velocities, when flowing full, of not less than 0.6 m/s or greater than 4.5 m/s based on Kutter’s or Manning’s formula using a “n” value of 0.013. Use of other practical “n” values may be permitted by the DOEC if deemed justifiable. Velocities above 4.5 m/s may be permitted with high velocity protection.

Table 5.2 details the minimum slopes, which will provide a velocity of 0.6 m/s when sewers are flowing full:

Table 5.2
Minimum Sewer Slopes

Minimum Slopes for Full-Pipe Velocity of 0.6 m/s	
Sewer Size (mm)	**Minimum Slope per 100 metres**
200	0.40
250	0.28
300	0.22
350	0.17
375	0.15
400	0.14
450	0.12
525	0.10
600	0.08
675	0.067
750	0.058
900	0.046

It is recommended that the actual pipe slopes should not be less than 0.5%.

To achieve 0.6 m/s flow velocities in sewers, which will flow less than ⅓ full, steeper slopes than given above must be used where conditions permit. For instance, the minimum slopes mentioned above would have to be doubled when depth of flow is only $^{1}/_{5}$ full and quadrupled when depth of flow is only $^{1}/_{10}$ full to achieve 0.6 m/s flow velocity.

Under special conditions, if full and justifiable reasons are given, slopes slightly less than those required for the 0.6 m/s velocity when flowing full may be permitted. Such decreased slopes will only be considered where the depth of flow will be 0.3 of the diameter or greater for design average flow. Whenever such decreased slopes are selected, the design engineer must include with their report, the computations of the anticipated flow velocities of average and daily or weekly peak flow rates. The pipe diameter and slopes shall be selected to obtain the greatest practical velocities to minimize settling problems. The operator of the sewer system will give written assurance to the DOEC that any additional sewer maintenance required by reduced slopes will be provided.

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

5.2.5.3. Sewers on Steep Slopes

Sewers on 20% slopes or greater shall be secured with concrete anchors, or equivalent, spaced as follows:

1. Grades 20% and up to 35% - not over 11 m centre to centre;

2. Grades 35% and up to 50% - not over 7.5 m centre to centre; and
3. Grades 50% and over - not over 5.0 m centre to centre.

5.2.5.4. Sewers in Tidal Zones

Sewers located within tidal zones and under the influence of rising and falling tides must be fitted with a sewage pumping station at the outfall. This is necessary to ensure proper functioning of the sewer system, including adequate flushing, reduction in freezing potential, and problems associated with tidal surge.

5.2.6. Sewer Location

5.2.6.1. Cross-Connection Prohibited

There shall be no physical connections between a public or private potable water supply system and a sewer, or appurtenances thereto which would permit the passage of any wastewater or polluted water into the potable supply. No water pipe shall pass through or come into contact with any part of a sewer manhole.

5.2.6.2. Relation to Water Works Structures

Sewers located in the vicinity of wells or other public water supply or structures should be constructed to conform to regulations governing public water supplies regarding minimum separation distances.

All existing waterworks units, such as basins, wells or other treatment units, within 60 m of the proposed sewer shall be shown on the engineering plans.

Soil conditions in the vicinity of the proposed sewer within 60 m of water supply sources shall be determined and shown on the engineering plans.

5.2.6.3. Relation to Watermains

5.2.6.3.1. Horizontal and Vertical Separation

Refer to Section 3.7.6.

5.2.6.3.2. Crossings

Refer to Section 3.7.6.2.

5.2.6.4. Stream Crossings

5.2.6.4.1. Cover Depth

The top of all sewers entering or crossing streams shall be at sufficient depth below the natural bottom of the streambed to protect the sewer lines. In general, the following cover requirements must be met:

1. 300 mm of cover where the sewer is located in rock;

2. 900 mm of cover in other material. In major streams, more than 900 mm of cover may be required; and

3. In paved stream channels, the top of the sewer line should be placed below the bottom of the channel pavement.

5.2.6.4.2. Horizontal Location

Sewers located along streams shall be located outside of the streambed and sufficiently removed from there to provide for future possible stream widening and to prevent pollution by siltation during construction.

5.2.6.4.3. Structures

The sewer outfalls, headwalls, manholes, gate boxes, or other structures shall be located so they do not interfere with the free discharge of flood flows of the stream.

5.2.6.4.4. Alignment

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

5.2.6.4.5. Materials

Sewers entering or crossing streams shall be constructed of ductile iron pipe with mechanical joints, or similar construction to ensure watertight joints free from change in alignment or grade. Material used to backfill the trench shall be stone, coarse aggregate, washed gravel, or other materials which will not readily erode, cause siltation, damage pipe during placement, or corrode the pipe.

5.2.6.4.6. Siltation and Erosion

Construction methods that will minimize siltation and erosion shall be employed. The design engineer shall include in the project specifications the method(s) to be employed in the construction of sewers in or near streams. Such methods shall provide adequate control of siltation and erosion by limiting unnecessary excavation, disturbing or uprooting trees and vegetation, dumping of soil debris, or pumping silt-laden water into the stream. Specifications shall require that cleanup, grading, seeding and planting or restoration of all work areas shall begin immediately. Exposed areas shall not remain unprotected for more than seven days.

5.2.7. Sewer Alignment

Sewers 600 mm diameter or less shall be laid with straight alignment and uniform grade between manholes. The alignment shall be checked by either using a laser beam or lamping. Where street layouts are such that straight alignment between manholes is impractical, sewers may be "curved" to conform to street curvature by the deflection of straight pipe in accordance with the manufacturer's recommendations. Curved sewers shall be limited to simple curves, which start and end at manholes. The minimum radius of curvature shall be 30 m. An alignment test such as "balling" must be conducted on curved sewers. When curved sewers are proposed, minimum slopes indicated in Section 5.2.5.2 must be increased accordingly to provide a recommended minimum velocity of 0.6 m/s when flowing full. Provision shall be made to provide additional maintenance for curved sewers.

5.2.8. Depth of Sewers

All sewers shall be laid at depths sufficient to drain basements and to be protected against damage by frost and traffic. Where, for specific reasons, shallow depths are necessary and can be justified, the sewer shall be protected to prevent damage by frost or traffic and insulated to prevent freezing. Sewers laid in deep or excessively wide trenches shall be adequately reinforced to prevent damage.

It is recommended that sewers be laid at a depth of at least 2.1 m. A minimum depth of 1.5 m may be permitted under some circumstances. Local soils and weather must be taken into account at all times.

5.2.9. Sewer Material

Any material generally accepted for sewers will be given consideration, but the material selected shall be suitable for the local conditions, such as the chemical characteristics of the sewage, character of industrial wastes, possibility of septicity, soil characteristics, exceptionally heavy external loadings, abrasion, corrosion, the necessity for reducing the number of joints, soft foundations, and other similar problems.

Suitable couplings complying with ASTM specifications shall be used for jointing dissimilar materials. The leakage limitations on these joints shall be in accordance with Section 5.2.11.

All sewers shall be designed to prevent damage from super-imposed live, dead and frost induced loads. Proper allowances for loads on the sewer shall be made because of soil and potential groundwater conditions, as well as the width and depth of trench.

Where necessary, special bedding, haunching and initial backfill, concrete cradle, or other special construction shall be used to withstand anticipated potential superimposed loading or loss of trench wall stability. See ASTM D2321 or ASTM C 12 when appropriate.

For new pipe materials for which ASTM standards have not been established, the design engineer shall provide complete pipe specifications and installation specifications developed on the basis of criteria adequately documented and certified in writing by the pipe manufacturer to be satisfactory for the specific detailed plans.

5.2.9.1. Warning/marker and Detection Tape

Warning/marker and detection tape as specified in the Department of Municipal and Provincial Affairs Water, Sewer and Roads Master Specification Section 02223.2.1 and detailed drawings numbered 0290 and 0300, shall be installed continuously with a minimum 1.0 m overlap at joints above water, sewer, and forcemains. Warning/marker tape shall be heavy gauge polyethylene, 150 mm wide and indicate the service line below. Detectable tape shall be either fabricated of detectable metallic material for underground installation or corrosion resistant insulated wires embedded in warning/marker tape. Detection tapes are intended for pipe location and must be installed above the pipe at an elevation 300 mm below ground surface and be detectable using conventional pipe location apparatus.

5.2.10. Overflow Structures and Bypasses

Backflow-preventing devices shall be provided whenever flooding of the sewer outlet may be possible.

The locations of bypasses shall be shown clearly on the plans and adequate details presented in order that the operation of such bypasses may be evaluated.

5.2.11. Sewer Testing and Inspection

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

Each section of a sewer, and its related appurtenances, shall be flushed prior to testing. The following tests are recommended as applicable.

5.2.11.1. Exfiltration Test

The exfiltration test shall be conducted as follows:

1. Fill test section with water in such a manner as to allow displacement of air in the line;

2. Immediately prior to test period add water to pipeline until there is a head of 1 m over the interior crown of the pipe measured at the highest of the test section or water in the manhole is 1500 mm above static ground water level, whichever is greater;

3. Duration of exfiltration test shall be 1 hour; and

4. Water loss at end of test period shall not exceed maximum allowable exfiltration over any section of pipe between manholes.

5.2.11.2. Infiltration Test

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

The infiltration test shall be conducted as follows:

1. Install a watertight plug at upstream end of pipeline test section;

2. Discontinue pumping operations for at least 3 days before test measurements are to commence and during this time keep thoroughly wet at least one third of pipe invert perimeter;

3. Prevent damage to pipe and bedding material due to flotation and erosion;

4. Place a 90° V-notch weir, or other measuring device approved by the DOEC in invert of sewer at each manhole; and

5. Measure rate of flow over a minimum of 1 hour, with recorded flows for each 5 minute interval

5.2.11.3. Allowable Leakage
Allowable leakage shall be determined by the following formula:

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

where: L = allowable leakage (L/hr);
D = diameter (mm);
S = Length of section (m); and
F = leakage factor (L/hr/mm of diameter/100 m of sewer).

Exfiltration Test:

Porous Pipe F = 0.12 L
Non-Porous Pipe F = 0.02 L

Infiltration Test:

Porous Pipe F = 0.10 L
Non-Porous Pipe F = 0.02 L

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

The test section shall be filled with air until a constant pressure of 28 kPa is reached. After a 2-minute period the air supply shall be shut off, and the pressure decreased to 4 kPa. The time required for the pressure to reach 17 kPa shall be measured.

5.2.11.5. Allowable Time for Air Pressure Decrease
Minimum times allowed for air pressure drop are provided in Table 5.3:

Table 5.3
Minimum Times Allowed for Pressure Drop

Pipe Diameter (mm)	Minimum Time (min:sec)
100	1:53
150	2:50
200	3:47
250	4:43
300	5:40
375	7:05
450	8:30
525	9:55
600	11:20

5.2.11.6. Sewer Inspection
The specifications shall include a requirement for inspection of manholes and sewers for watertightness, prior to placing into service.

Video inspections shall be conducted in accordance with the *Municipal Water, Sewer and Roads Specifications*.

5.2.12. Sanitary Sewer Service Connections
Sewer services shall be consistent with the local or provincial plumbing and drainage regulations. It is required that, unless tees or Y's have been installed, saddles be used in connecting the service pipe to the sewer. Generally, these are placed at an angle of 45° above the horizontal. If a saddle type connection is used, it shall be a device designed to join with the types of pipe, which are to be connected. All materials used to make service connections shall be compatible with each other and with the pipe materials to be joined and shall be corrosion proof. Connections shall be made by qualified personnel only.

5.2.13. Manholes
Sewer manholes are mainly for the purpose of facilitating maintenance and operation of the sewer system and shall be designed, constructed and located as follows:

5.2.13.1. Minimum Diameter
Minimum diameter for manholes shall be 1200 mm; larger diameters are preferred for larger sewers.

5.2.13.2. Manhole Covers

Minimum cover clear opening shall be 580 mm. Watertight covers should be used where manholes will be subject to flooding. Where significant sections of sewers are provided with watertight manholes, extended vents may be required to prevent excessive sulphide generation. Locked manhole covers may be desirable in isolated easement locations or where vandalism may be a problem.

5.2.13.3. Location

Manholes shall be located at all junctions, changes in grade, size or alignment (except with curvilinear sewers) and termination points of sewers.

The maximum acceptable spacing for manholes is 90 to 120 m for sewers 200 to 450 mm in diameter. Spacings of up to 150 m may be used for sewers 450 mm to 750 mm in diameter. Larger sewers may use greater manhole spacing.

Cleanouts may be used only with approval from DOEC and shall not be substituted for manholes nor installed at the end of laterals greater than 45 m in length.

5.2.13.4. Drop Manholes

Drop manholes should be provided for lateral sewers entering a manhole at an elevation of 600 mm or more above the manhole invert. Where the difference between the incoming sewer and the manhole invert is less than 600 mm, the invert should be filleted to prevent solid deposition.

Drop manholes shall be constructed with an outside or inside drop connection. Inside drop connections shall be secured to the interior wall of the manhole and provide access for cleaning as per the *Municipal Water, Sewer and Roads Specifications*.

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

5.2.13.5. Channel and Benching

The flow channel through manholes should be made to conform in shape and slope to that of the sewers. The channel walls should be formed or shaped to the full height of the crown of the outlet sewer in such a manner to not obstruct maintenance, inspection or flow in the sewers.
When curved flow channels are specified in manholes, including branch inlets, minimum slopes indicated in Section 5.2.5.2 should be increased to maintain acceptable velocities.

A bench shall be located on each side of any manhole channel when the pipe diameter(s) are less than the manhole diameter. The bench should be sloped no less than 4%. The direct connection of sewer service lines to manholes is prohibited unless the service enters at the flow line of the manhole.

5.2.13.6. Manhole Steps

Manhole rung spacing shall be not less than 300 mm or more than 375 mm. Their minimum width should be 375 mm and they should be designed to prevent the foot from slipping sideways. Adequate clearance should be provided between the wall of the manhole and the rungs to afford secure footing.

5.2.13.7. Watertightness

Manholes shall be of the pre-cast concrete or poured-in-place concrete type. Manholes shall be waterproofed on the exterior. Manhole lift holes and grade adjustment rings shall be sealed with non-shrinking mortar or other acceptable material.

Inlet and outlet pipes shall be joined to the manhole with a gasketed, flexible, watertight connection or any watertight connection arrangement that allows differential settlement of the pipe and manhole wall to take place.

Watertight manhole covers are to be used wherever the manhole tops may be flooded by street runoff or high water. Locked manhole covers may be desirable in isolated easement locations or where vandalism may be a problem.

5.2.13.8. Frost Lugs

Where required, frost lugs should be provided to hold pre-cast manhole sections together.

5.2.13.9. Safety Chains

Safety chains should be provided on the downstream side of manholes for sewers 1200 mm in diameter or greater.

5.2.13.10. Inverted Siphons

The use of inverted siphons shall be kept to a minimum, but where they must be used, they shall consist of at least two lines, one of which shall be not less than 200 mm in diameter. Inverted siphons should be designed in size and grade to maintain a velocity of at least l m/s under conditions of average dry weather flow. Under minimum dry weather flow, the independent operation of one of the lines shall provide a minimum velocity of l m/s. Where the above conditions cannot be met, other means shall be provided. They shall be provided with necessary appurtenances for maintenance, convenient flushing and cleaning equipment. The inlet and discharge structures shall have adequate clearances for cleaning equipment, inspection and flushing. The inlet and outlet details shall be arranged so that the normal flow is diverted to one barrel and so that either barrel may be cut out of service for cleaning. The vertical alignment should permit cleaning and maintenance.

5.2.14. Alternative Wastewater Collection Systems

The use of alternative wastewater collections systems will be considered only when the conventional wastewater system is either impractical or cost prohibitive. The consultant will have to convince the DOEC regarding the viability and long-term sustainability of the proposed alternative wastewater system.

5.2.14.1. Applications

Under a certain set of circumstances, each alternative system has individual characteristics, which may dictate standards for usage. Each potential application should be analyzed to determine which system is most cost effective and which will comply with local requirements. The following features of various sewerage alternatives are considered in a planning a project.

5.2.14.2. Population Density

Conventional sewers are typically costly on a lineal meter basis. When housing is sparse, resulting in long reaches between services, the cost of providing conventional sewers is often prohibitive. Pressure sewers, small diameter gravity sewers and vacuum sewers are typically less costly on a lineal meter basis, and often prove to be more cost-effective when serving sparse populations.

5.2.14.3. Ground Slopes

Where the ground profile over the main slopes continuously downward in the direction of flow, conventional or small diameter gravity sewers are normally preferred. If intermittent rises in the profile occur, conventional sewers may become cost prohibitive. The variable grade gravity sewer variation of small diameter gravity sewers, by use of inflective gradients and in conjunction with *septic tank effluent pump* (STEP) pressure sewer connections, can be economically applied. Vacuum sewers may be particularly adaptable to this topographic condition, so long as head requirements are within the limits of available vacuum. In flat terrain conventional sewers become deep due to the continuous downward slope of the main, requiring frequent use of lift stations. Both the deep excavation and the lift stations are expensive. *Small Diameter Gravity Sewers* (SDGS) are buried less deep, owing to the flatter gradients permitted. Pressure sewers or vacuum sewers are often found to be practical in flat areas, as ground slope is of little concern. In areas where the treatment facility or interceptor sewer are higher than the service population, pressure sewers and vacuum sewers are generally preferred, but should be evaluated against SDGS systems with lift stations.

5.2.14.4. Subsurface Obstacles

Where rock excavation is encountered, the shallow burial depth of alternative sewer mains reduces the amount of rock to be excavated. Conventional sewers require deep excavation, and may sometime encounter groundwater. Depending on the situation, dewatering can be expensive and difficult to accomplish.

5.2.14.5. Low Pressure Collection Systems

In certain areas primarily due to the very rocky terrain servicing using conventional gravity sewers has been found to be economically unacceptable. An alternative to the use of gravity sewers is the use of a pressure sewer system.

5.2.14.5.1. Definition

A pressure sewer system uses small diameter, when compared with a conventional gravity sewer, polyethylene pressure pipe to convey sewage to a central treatment or collection point. Each property discharges into a common pressure pipe using a specially designed grinder pump, usually installed in the basement or crawl space of the house.

5.2.14.5.2. Design

5.2.14.5.2.1. Grinder Pump

The design of a pressure sewer system is totally dependent upon the efficiency and reliability of the grinder pump. The pump has to be designed for this specific application and must be capable of the following:

1. Grinding the waste to a fine slurry to enable pumping through small diameter mains. The pump must be capable of routinely handling items which are commonly found in domestic sewage, for example, plastic, wood, rubber and light metal objects;

2. As each pump will be located at a different point along the common pressure main, at various elevations and might operate either individually or in unison with several other pumps, it is essential that it be able to operate consistently over a very wide range of heads which are continually and often rapidly changing; and

3. Capability of operating at least 25% above the low-pressure sewer system design criteria of 280 kPa. This is based upon the maximum daily number of pumps operating simultaneously. The simultaneous operation of more than the design maximum number of pumps is usually a transitory occurrence, however, it is essential that no damage occur to the pumping equipment, pipelines or appurtenances.

5.2.14.5.2.2. Storage Tank

The storage tank is to be sized to provide a pump "on-off" cycle most desirable for the efficient and durable operation of the pump. The sizing should minimize the potential for the generation of septic sewage due to long retention periods.

5.2.14.5.2.3. Collection Pipe

As required by the Department of Municipal and Provincial Affairs, *Municipal Water, Sewer and Roads Specifications*.

5.2.14.5.2.4. Frost Protection

The majority of pressure sewer systems are constructed in shallow trenches, to minimize the quantity of rock excavation. Adequate protection must be provided to prevent freezing using a combination of insulation and heat tracing. The method of pipe installation must be of proven design for the specific application.

5.2.14.5.2.5. Common Trench Construction

The requirements of Section 5.3.20.6 must be adhered to.

5.2.14.5.3. Applications for Approval

All applications for the approval of a pressure sewer system should contain, as well as the information described in Section 1.0, the following additional information:

1. A detailed cost analysis of the pressure sewer system alternative as compared to conventional servicing, including:
   a) Capital cost;
   b) Purchase and installation cost of the individual pump units; and
   c) Operation and maintenance costs.

2. The municipality in which the system is to be installed must agree with the proposal to utilize pressure sanitary sewers;

3. The municipality must indicate that it has either:
   a) Adequately trained personnel on staff for the purpose of repair and maintenance of the individual pumping units and the collection system; or
   b) Has executed an operative/maintenance agreement with a local company, which has trained personnel in this respect.

4. Approval from each homeowner to permit access to the pump unit for servicing, when it is contained within the house. (Note: It is possible to house the pump unit within a chamber external to the house, however, this adds to the capital cost of the system); and

5. Complete details of the type of pump to be used including complete performance data and proof of reliability for this specific application. The pump supplier must maintain an adequate supply of spares and replacement units in the Province of Newfoundland and Labrador.

5.2.14.6. Vacuum Sewer Systems

Vacuum sewer systems consist of a vacuum station, collection piping, wastewater holding tanks, and valve pits. In these systems, wastewater from an individual building flows by gravity to the location of the vacuum ejector valve. The valve seals the line leading to the main in order to maintain required vacuum levels. When a given amount of wastewater accumulates behind the valve, the valve opens and then closes allowing a liquid plug to enter the line. Vacuum pumps in a central location maintain the vacuum in the system.

5.2.14.6.1. Services

Each home on the system should have its own holding tank and vacuum ejector valve. Holding tank volume is usually 115 L. As the wastewater level rises in the sump, air is compressed in a sensor tube, which is connected to the valve controller. At a preset point, the sensor signals for the vacuum valve to open. The valve stays open for an adjustable period of time and then closes. During the open cycle, the holding tank contents are evacuated. The timing cycle is field adjusted between 3 and 30 seconds. This time is usually set to hold the valve open for a total time equal to twice the time required to admit the wastewater. In this manner, air at atmospheric pressure is allowed to enter the system behind the wastewater. The time setting is dependent on the valve location since the vacuum available will vary throughout the system, thereby governing the rate of wastewater flow.

The valve pit is typically located along a property line and may be combined with the holding tank. These pits are usually made of fibreglass, although modified concrete manhole sections have been used. An anti-flotation collar may be required in some cases.

5.2.14.6.2. Collection Piping

The vacuum collection piping usually consists of 100 mm and 150 mm mains. Smaller 75 mm mains are not recommended as the cost savings of 75 mm versus 100 mm mains are considered to be insignificant.

Rubber gasketed PVC pipe, which has been certified by the manufacturer as being suitable for vacuum service is recommended. Solvent welding should be avoided when possible. The mains are generally laid to the same slope as the ground with a minimum slope of 0.2 percent. For uphill transport, lifts are placed to minimize excavation depth. There are no manholes in the system; however, access can be gained at each valve pit or at the end of a line where an access pit may be installed. Installation of the pipe and fittings follows water distribution system practices. Division valves are installed on branches and periodically on the mains to allow for isolation when troubleshooting or when making repairs. Plug valve and resilient wedge gate valves have been used.

5.2.14.6.3. Vacuum Station

Vacuum stations are typically two-story concrete and block buildings approximately 7.5 m x 9 m in floor plan. Equipment in the station includes a collection tank, a vacuum reservoir tank, vacuum pumps, wastewater pumps, and pump controls. In addition, an emergency generator is standard equipment, whether it is located within the station, outside the station in an enclosure, or is of the portable, truck mounted variety.

The collection tank is made of either steel or fibreglass. The vacuum reservoir tank is connected directly to the collection tank to prevent droplet carryover and to reduce the frequency of vacuum pump starts. Vacuum pumps can be either liquid ring or sliding vane type and are sized for a 3 - 5 hour/day run-time. The wastewater discharge pumps are non-clog pumps with sufficient net positive suction head to overcome tank vacuum. Level control probes are installed in the collection tank to regulate the wastewater pumps. A fault monitoring system alerts the system operator should a low vacuum or high wastewater level condition occur.

5.2.14.7. Small Diameter Gravity Sewers

Small diameter gravity sewers (SDGS) require preliminary treatment through the use of an interceptor or septic tanks upstream of each connection. With the solids removed, the collector mains need not be designed to carry solids, as conventional sewers must be. Collector mains are smaller in diameter and are laid with variable or inflective gradients. Fewer manholes are used and most are replaced with cleanouts except at major junctions to limit infiltration/inflow and entry of grit. The required size and shape of the mains is dictated primarily by hydraulics rather than solids carrying capabilities.

5.2.14.7.1. House Connections

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

5.2.14.7.2. Interceptor Tanks

Interceptor tanks are buried, watertight tanks with baffled inlets and outlets. They are designed to remove both floating and settleable solids from the waste stream through quiescent settling over a period of 12-24 hours. Ample volume is provided for storage of the solids, which must be periodically removed through an access port. Typically, a single-chamber septic tank, vented through the house plumbing stack vent, is used as an interceptor tank.

5.2.14.7.3. Service Laterals

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

5.2.14.7.4. Collector Mains

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

5.2.14.7.5. Cleanouts, Manholes, and Vents

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

5.2.14.7.6. Lift Stations
Lift stations are necessary where the elevation differences do not permit gravity flow. Either STEP units (see Section 5.2.14.3) or mainline lift stations may be used. STEP units are small lift stations installed to pump wastewater from one or a small cluster of connections to the collector main, while a mainline lift station is used to service all connections in a larger drainage basin.

5.2.15. Outfall Sewers

5.2.15.1. Approval
The location and discharge from an outfall sewer could have a major impact on the environment. The selection of the various parameters to be used for the design of an outfall would be included in the Technical Report (Section 2).

Site-specific studies may be required where:
1. A municipal system collects and processes industrial waste;
2. Effluent discharges to a fresh water body;
3. Municipal waste or effluent discharges to sensitive marine environment; and
4. Any other situation as deemed appropriate by the DOEC.

5.2.15.2. Design
The objective of an outfall is to introduce the effluent stream into the receiving water in a place and manner chosen to achieve efficient mixing with that receiving water. Outfalls must be located, designed, constructed and maintained for efficient mixing of discharges with receiving waters. The design must be based on the analysis of salinity, temperature and dissolved oxygen profiles, wind speed and direction, tidal currents, and other relevant oceanographic measurements, and must take into consideration both positive and negative buoyant plumes. An outfall shall not impact on any intake for fish plants, shellfish beds, aquaculture areas, recreational areas, and any other sensitive areas.

5.2.15.2.1 Length and Depth
Outfall lengths, depths, and depth-distance combinations for marine discharges of municipal wastewater, are given in Table 5.4. A length-depth combination less than that specified in Table 5.4 may be permitted where the proposed treatment and site-specific studies can demonstrate that acceptable dilutions can be attained to meet the receiving water quality guidelines.

5.2.15.2.2. First Point of Discharge
The distance to the first point of discharge from the mean low water mark must be equal to or greater than 30 m.

5.2.15.2.3. Protection and Maintenance
The outfall sewer shall be designed and constructed to protect against the effects of floodwater, tides, ice or other hazards as to reasonably insure its structural stability and freedom from stoppage. A manhole should be provided at the shore end of all gravity sewers extending into the receiving waters. Hazards due to navigation must be considered in designing outfall sewers.

Table 5.4
Depth and Length Requirements for Marine Outfalls

Discharge Rate (Q) (Litres/day)		Depth and Length for Outfalls (m)		
		Shellfish, Intakes and Aquaculture	*Recreational*	*Other*
Q # 50,000	Depth Length	Study Required	5 50	3 30
50,000 < Q # 350,000	Depth Length	Study Required	6 75	5 50
>350,000	Depth Length	Study Required	Study Required	Study Required

Length refers to the distance from the low water mark to the point of discharge or the first diffuser nozzle.
* Depth refers to the distance between the low normal spring tide level to the top of the outfall.

5.2.15.2.4. Signage

Signs are to be posted to indicate the location of outfalls within 100 m.

5.2.15.2.5. Offset from Bottom

An appropriate offset is to be maintained between the discharge point and bottom of the receiving water body, to ensure that plume is not affected by the bed of the water body and vice versa.

5.2.15.2.6. Sampling Provisions

All outfalls shall be designed so that a sample of the effluent can be obtained at a point after the final treatment process and before discharge to or mixing with the receiving water.

5.2.15.3. Receiving Water Quality Objectives

The typical level of treatment required for any new treatment plant in the province is secondary treatment with disinfection. However, each new plant will be evaluated on a case-by-case basis. Required levels of treatment may be determined to be higher or lower than secondary treatment based on waste assimilation studies. The procedure for carrying out these studies is described in the following section.

5.2.15.3.1. Assimilation Study Procedures

1. Level of Effort - As part of the pre-design evaluation, the engineer shall determine in consultation with DOEC the level of effort required for the particular wastewater assimilation study. The department may conclude that the effects of the proposed project on the receiving water will be minimal. In this case, the department will set effluent limitations based upon a simple model (possibly basic dilution calculations). In this case, only a minimal level of effort is required for the receiving water study (RWS). The department will determine which parameters will require measurement.

When the department is unsure of the possible effects of a project on the receiving water, it may require that an intermediate RWS and model simulation be conducted as a preliminary assessment tool. If the results of this study indicate that the proposed project would have only a minor effect on the receiving water, the department may, at that point, set effluent limitations. The department shall set the data requirements for the intermediate RWS's.

The third level of effort that may be required is a detailed RWS and complex modelling application. The results of this procedure will determine required effluent limitations. This approach will be required when the department believes that a proposed project may have a significant impact on the receiving water quality. The department will determine RWS data requirements.

2. Water Sampling Procedures - Instruments for electronic *in-situ* determination of water quality parameters should be calibrated at least before and after each sampling trip. For example, samples should be collected for salinity to verify field measurements and samples fixed in the field for dissolved oxygen to verify dissolved oxygen probes.

   All field collection equipment should be listed and prepared before each sampling trip, insuring that all collection containers are clean and proper log forms and labelling equipment are available. Different containers should be available for metals, nutrients, organics, dissolved oxygen, etc. due to their cleaning and preservation requirements.

   An established sequence of collection should be developed and maintained throughout the monitoring effort, insuring that new personnel are trained in the proper methods and sequence of data collection. All samples should be logged and sample log sheets should include station location, time, depth, results of *in situ* sampling, and container numbers for each type of sample. Datum should always be clearly specified (e.g. time of day standard, datum for water surface elevations).

   All samples should be preserved on board, where the preservation technique will vary with the type of analysis required, but may involve icing, acidification, organic extraction, etc. The preservation techniques should be documented prior to implementation to the monitoring study. For some samples that do not preserve well it may be necessary to either conduct analyses on board or quickly transfer them to nearby on-shore facilities.

   Additional samples should be collected to determine sampling variability and individual samples may be split prior to analysis to determine analytical variability. The number of replicate samples should be established as part of the planning for the monitoring effort. Field samples may also be spiked with a known amount of a standard prior to analysis. The identity of the spiked, split and duplicate samples should be kept on separate logs and the analyst should not be aware of their identity.

   The samples should be transferred from the field to the laboratory in a timely manner. The field logs should be recorded and a laboratory log kept of the samples and their arrival. Custody sheets may be kept to further document the transferral of samples.

5.2.15.3.2. Assimilation Capacity of Receiving Water

In essence, the dilution/assimilative capacity of a waterbody depends on wastewater characteristics and a host of physical, chemical and biological factors, such as the flow or volume of the waterbody and the wastewater discharges, dispersion of effluent, depth and width of the waterbody, type of substrate, algal growths, benthic deposits or organic sludges, etc.

A wastewater assimilation study is the mechanism to be used in estimating a waterbody's assimilative capacity and establishing effluent requirements to meet the Canadian Environmental Quality Guidelines (CEQG). Either simple dilution formulae or more sophisticated mathematical models can be used as assessment techniques, depending on the circumstances. For example, with a dilution ratio greater than 20 to 1, simple dilution formulae may be adequate for estimating effluent requirements for discharges with a high degree of treatment (e.g. secondary treatment) and which do not contain hazardous substances. With a dilution ratio less than 20 to 1, more complex assessment techniques may be required to estimate assimilative capacity. Further, under complex situations (e.g. multiple uses of water, flood control requirements, etc.) sophisticated mathematical models may be used to estimate assimilative capacity and effluent requirements.

In areas with existing water quality better than CEQG, it is a good general principle not to allocate the entire assimilative capacity of a receiving waterbody. The need for maintenance of a reserve capacity should be established on a case-by-case basis.

In addition to meeting the CEQG, a thorough receiving water assessment may be required before the discharge of effluent containing toxic substances will be permitted. Such an assessment should include studies of the potential accumulation and concentration of the substances in the environment (such as bed sediments and aquatic flora and fauna), synergistic effects with other substances and physical factors (such as temperature changes or radiant energy) that may affect the environmental impact of contaminants.

5.2.15.3.2.1. Dilution Ratio

Dilution ratio is a simple measure of a receiving water's assimilative capacity. Dilution ratios should be based upon the 7 consecutive day average low streamflow occurring once in 20 years (7Q20), and the peak hourly effluent discharge rate (both expressed in the same units).

5.2.15.3.2.2. Mixing Zone

A mixing zone is a region of a waterbody in which an effluent discharge with quality (chemical/physical/biological) characteristics different from those of the receiving water is in transit and is progressively assimilated from the immediate outfall area to the outer limits of the region. At the boundaries or outer limits of the mixing zone, water quality objectives established by the DOEC to protect beneficial water uses should be achieved. A mixing zone may not be used as an alternative to adequate treatment. Existing biological, chemical, physical and hydrological conditions should be known when considering the location of a new mixing zone or limitations on an existing one.

No conditions within the mixing zone should be permitted which:

1. Are rapidly lethal to important aquatic life (resulting in conditions which result in sudden fish kills and mortality of organisms passing through the mixing zones);
2. Cause irreversible responses which could result in detrimental postexposure effects;
3. Result in bioconcentration of toxic materials which are harmful to the organism or its consumer; or
4. Attract organisms to the mixing zones, resulting in a prolonged and lethal exposure period.

The mixing zone should be designed to satisfy the following conditions:
1. Shall allow an adequate zone of passage for the movement or drift of all stages of aquatic life (specific portions of a cross-section of flow or volume may be arbitrarily allocated for this purpose);
2. Shall not interfere with the migratory routes, natural movements, survival, reproduction (spawning and nursery areas), growth, or increase the vulnerability to predation, of any representative aquatic species, or endangered species;
3. Eliminate rapid changes in the water quality, which could kill organisms by shock effects;
4. Total loading from all mixing zones within a waterbody must not exceed the acceptable loadings from all point source discharges required to maintain satisfactory water quality;
5. Mixing zones should not result in contamination of natural sediments so as to cause or contribute to exceedances of the water quality objectives outside the mixing zone

The mixing zone shall be:
1. Free from substances in concentrations or combinations which may be harmful to human, animal or aquatic life;
2. Free from substances that will settle to form putrescent or otherwise objectionable sludge deposits, or that will adversely affect aquatic life or waterfowl;
3. Free from debris, oil, grease, scum or other materials in amounts sufficient to be noticeable in the receiving water;
4. Located so as not to interfere with fish spawning and nursery areas;
5. Free from colour, turbidity or odour-producing materials that would:
   a) Adversely affect aquatic life or waterfowl;
   b) Significantly alter the natural colour of the receiving water;
   c) Directly or through interaction among themselves or with chemicals used in water treatment, result in undesirable taste or odour in treated water, and;
   d) Free from nutrients in concentrations that create nuisance growths of aquatic weeds or algae or that results in an unacceptable degree of eutrophication of the receiving water;

5.2.15.3.2.2.1. Calculating the Mixing Zone

The mixing zone should be as small as practicable, and shall not be of such size or shape to cause or contribute to the impairment of existing or likely water uses. Mixing zone size shall be established on a case-by-case basis, but in no case shall it exceed the following:

For marine bodies of water the following, measured from the point of discharge and from the mean low water mark, apply for the purpose of calculating the mixing zone:

1. The depth/height is the distance from the bed to the surface of the water; and

2. The radius is, either:
   a) 100 m; or
   b) 25% of the width of the body of water, whichever is less.

For an estuary the following, measured from the point of discharge and from the mean low water mark, apply for the purpose of calculating the mixing zone:

1. The depth/height is the distance from the bed to the surface of the body of water;

2. The width, perpendicular to the path of the stream, is the lesser of:
   a) 100 m; or
   b) 25% of the width of the stream or estuary; and

3. The length, parallel to the path of the stream, is the distance between a point 25 metres upstream and a point, which is the lesser of:
   a) 175 m downstream; or
   b) A distance downstream at which the width of the effluent plume equals the width determined above for an estuary (Section 5.2.15.3.2.2.1 (2)).

5.2.15.3.3. Waste Assimilation Study Field Procedures for Coastal Waters

5.2.15.3.3.1. General Strategies

1. Before any fieldwork is carried out on any coastal water, the problem(s) should first be defined, and the objective(s), of the study laid out.

2. Maps and hydrographic charts of the proposed discharge area shall be obtained, as well as any previous reports on the waterbody and any nearby municipal and/or industrial discharges.

3. All existing data on water quality monitoring, water uses, current speed and direction, receiving water density distribution, and volumes and characteristics of waste discharges should be obtained.

5.2.15.3.3.2. Pre-Design Surveys vs. Monitoring Surveys

Oceanographic surveys for wastewater disposal systems can be placed in one of two general categories - predesign or monitoring. Each of the two types of surveys has a different objective and possesses unique requirements that demand careful consideration.

Predesign surveys must provide not only the necessary information to determine the proper alignment of the submarine outfall, the location and orientation of the wastewater diffuser system, but also the final design criteria for the outfall and diffuser system which will assure the protection and enhancement of the receiving environment. In addition, accurate bathometric profiles, benthic soil characteristics for outfall placement and foundation, and sediment erosion and deposition behaviour must be determined.

Monitoring surveys are of several types. Predischarge monitoring surveys are conducted to establish the baseline or natural conditions of the receiving water, receiving sediments and adjacent shoreline prior to discharge of wastewaters. Post discharge monitoring surveys are conducted for the purpose of determining the effects of the wastewater discharge on the receiving environment or for the purpose of assessing the performance of the disposal system for verification of design criteria and future design improvements.

5.2.15.3.3.3. Pre-Design Waste Assimilation Studies

5.2.15.3.3.3.1. General Objectives

Three major objectives must be satisfied in the conduct of a pre-design oceanographic survey. Firstly, a pre-design survey must determine the dispersion or diluting characteristics of the receiving water. Secondly, a pre-design survey must provide sufficient information on the ecosystem in the proposed discharge area to assure that biologically significant or sensitive areas will not be adversely affected by the disposal system, both during construction of the outfall and during the continuing discharge of the wastewater. Thirdly, foundation conditions for outfall and diffuser placement must be determined prior to preparation of engineering plans and specifications on the waste disposal system.

To satisfy all three objectives a general area for placement of the disposal system must be surveyed and potential alternative outfall sites chosen.

Because the rational determination of an outfall length necessary to meet particular receiving environment requirements for a specified level of treatment will be dependent upon the survey results, the pre-design oceanographic survey should be designed to provide the greatest practical flexibility in the selection of outfall alignments and lengths.

5.2.15.3.3.3.2. Measurement of Parameters

Those parameters, which are most critical to the design, should be most thoroughly measured in the conduct of the oceanographic survey, with less effort expended in assessing more traditional parameters, which have a much lesser effect on design considerations. The critical parameters include the following:

1. Density of Receiving Water - Density measurements throughout the water column are required in order to estimate the extent of initial dilution occurring over a diffuser. Water density may be determined by calculation from temperature and salinity, conductivity or specific measurements.

2. Horizontal Ocean Currents - Horizontal current velocity may be measured by using propeller or cone-type meters suspended at a specific depth.

3. Horizontal Eddy Dispersion - Measurement and prediction of the magnitude of horizontal eddy dispersion requires determination of horizontal ocean currents occurring in the horizontally moving wastewater field and of the appropriate eddy diffusivity. Measurements of the diffusivity, or diffusion coefficient, are very difficult and require sophisticated and complex techniques. The relative dilution effected by eddy dispersion of concern to most wastewater disposal systems, however, is small, permitting a rather gross estimate of the diffusivity without affecting the design substantially.

   The effect of wave climate on surface and alongshore currents should also be evaluated. The effect of ice accumulation around outfall pipes should also be analyzed.

4. Decay/Disappearance Rates - Determination of disappearance rates for specific non-conservative constituents requires, in most cases, special studies wherein a mass of the discharged wastewater containing the constituent is monitored in the receiving environment over a period of time to determine its decay as a function of time. The observed diminution must be corrected for physical dilution that has occurred by eddy dispersion over the period of observation. A tracer material, usually non-toxic fluorescent dye, should be employed in these studies to obtain physical dilution.

5. Wind Velocity and Direction

6. Receiving Water Quality Parameters - To assure placement of an outfall in an area which will provide the least deleterious effects on the environment and to predict what effects the wastewater discharge will have on the receiving environment, it is necessary to characterize the physical and chemical conditions of the receiving environment and the indigenous flora and fauna in the inter-tidal zones, in the benthic sediments and in the overlying waters.

   Water quality characterization should include dissolved oxygen, pH, temperature, salinity, transparency or turbidity, total and fecal coliforms, BOD, and in some cases, nitrogenous and phosphorous forms. Benthic sediments should be characterized with respect to particle size distribution, organic carbon and nitrogen, dissolved sulphide, heavy metals, and chlorinated hydrocarbons.

   Microplankton, macroplankton and nekton populations in the receiving waters may be sampled and their diversity determined. The benthos should be properly examined for specific enumeration, and the extent of biological productivity and diversity should be determined.

7. Benthic Soil - Prior to preparation of engineering plans and specifications the structural characteristics of the benthic soils must be determined. For ease and economics of construction of the outfall, the alignment should not encounter rock outcroppings, and other submerged obstructions, abrupt vertical discontinuities or escarpments. Coring and soil analysis should be performed to determine footing characteristics, and bathometric profiles

should be made over a period of time to determine if problems with shoaling and shifting sediments are likely to occur.

5.2.15.3.3.3.3. Survey Procedure

Because oceanographic surveys for pollution control facilities are usually restricted by economic and time constraints, it is extremely important to specify a survey program that will provide the most useful and meaningful data within the resources available, and should include the following:

1. Sampling Stations - Sampling station location should be selected to provide adequate coverage of the receiving environment and should be for the most part located at coordinates representing potential diffuser locations and outfall alignments. Sampling points for certain measurements within the water column should be selected to provide the best representative sample, or samples, of the entire column or of the characteristic under consideration.

2. Current Measurements - Synoptic current measurements throughout the receiving water mass over an extended period of time will provide the best description of the current structure and circulation pattern, but sufficient resources are generally not available for such extensive sampling. A reasonable assessment of a current regime at any particular time can be obtained by taking measurements of current speed and direction near the bottom and top of the water column, and at mid-depth. If the number of current metering points is further limited because of time or other constraints, single-depth current measurements will be most useful at a depth representative of the currents responsible for horizontal movement away from the diffuser following the initial dilution process. In those instances where a pronounced pyenocline exists, the best single depth for measurement would be several meters below the pyenocline. Where there is no such pyeriocline, single depth measurements at several meters below the surface would usually provide the most useful information.

   Coastal currents are affected by lunar tides, major oceanic currents, wind stress, and tributary freshwater discharges, which are all variable in magnitude and effect with respect to time. To obtain a reasonable estimate of the current velocity distribution, therefore, measurements should be taken over a sufficiently long period (preferably one year) to account for the diurnal and seasonal changes.

3. Water Quality Measurements - Water quality characteristics are less variable than current characteristics, but do vary somewhat diurnally and greatly seasonally. Thus, only one or two measurements of water quality characteristics are generally required during a single day several times during a year.

   Water quality characteristics can be measured either *in-situ* with direct reading or recording devices or from discrete collected samples. Existing equipment allows *in-situ* simultaneous measurement of DO, pH, transmittance, temperature and conductivity, and easily provides the necessary vertical definition to establish density gradients and pycnoclines. Other measurements, such as nutrient concentrations, must be made from discrete samples collected with an appropriate water-sampling device.

4. Biological Parameters - Of the biological parameters, the benthic flora and fauna are the least affected by diurnal and seasonal factors, and thus can be characterized adequately by sampling only several times during a year. Benthic samples can be collected either remotely from a vessel or directly by divers. Remote sampling is performed either with a dredge, which allows for recovery of a disturbed sample, or a coring tool, which provides a relatively undisturbed specimen. Generally, dredges are used for biological characterization of the sediments and corers are employed for physical and chemical assay of the benthic materials.

   Several samples are usually taken per time in order to provide an indication of localized sample variation.

   A more systematic and specific approach, but also more biased, to benthos characterization is provided by divers who can observe and report on general biological conditions and obtain samples and specimens for later analysis that are highly controlled with respect to size and location.

   Because the distributions of the most motile and the floating forms, e.g. microplankton, macroplankton and nekton, are highly time and space dependent, a large number of samples for these organisms must be collected to obtain a statistically significant characterization.

   Plankton sampling is usually accomplished by vertical or horizontal tows with appropriately sized netting. Discrete samples collected with conventional water samplers can also be used for plankton enumeration and identification, but this procedure allows for much greater sampling error due to the generally large spatial variations encountered in plankton populations and distributions.

5.2.15.3.3.3.4. Equipment
Current meters can be classed either as moored or non-moored, and can be direct reading or recording. Moored recording meters provide an almost continuous record of current velocity, but are restricted because each individual meter is fixed in both the vertical and horizontal plane. Meters operated from a vessel, on the other hand, provide measurements from a variety of depths and locations, but present only an instantaneous sampling of the current regime.

5.2.15.3.4. Waste Assimilation Study Field Procedures for Estuaries
1. An estuary is defined as the tidal mouth of a large river. Before any fieldwork is carried out on an estuary, the problem(s) should first be defined, and the objectives of the study laid out.

2. Maps and aerial photographs of the survey area should be obtained, as well as any previous reports on the waterbody and municipal and/or industrial discharges. All discharges to the waterbody should be pinpointed on the map. All existing data on water quality monitoring, water takings and consumption, water uses, flows, and volumes and characteristics of waste discharges should be obtained.

3. If manpower and time permit, the entire reach of the estuary to be studied should be inspected. Waste discharges, dispersion patterns of effluents, physical characteristics of the estuary, water uses, algal growths, the presence of benthic deposits or organic sludges and any other pertinent characteristics should be noted.

4. Boundary condition data are external to the model domain and are driving forces for model simulations. For example, atmospheric temperature, solar radiation and wind speeds are not modelled but are specified to the model as boundary conditions and drive modelled processes such as mixing, heat transfer, algal growth, reaeration, photolysis, volatilization, etc. Non-point and point source loadings as well as inflow water volumes are model boundary input. The boundaries at the upstream end of the estuary and the open boundary at the ocean provide major driving forces for change. Models do not make predictions for the boundary conditions but are affected by them.

5. In setting limits on wastewater quantity and quality, the following factors affecting estuarine water quality should be assessed: salinity, sediment, bacteria and viruses dissolved oxygen depletion, nutrient enrichment and over-production, aquatic toxicity, toxic pollutants and bioaccumulation and human exposure:
   a) Salinity is important in determining available habitat for estuarine organisms. Large wastewater discharges into relatively small estuaries or embayments can alter the local salinity regime through dilution. Even when the salinity is not affected by the discharge, it is measured and modelled in order to quantify advection and dispersion. These processes help determine how wastewater is assimilated into the estuary.

   b) Sediment enters estuaries from many sources, and can alter the habitat of benthic organisms. Sediment is also an important carrier of such pollutants as hydrophobic organic chemicals, metals, and nutrients.

      Sediment transport can move pollutants upstream, or between the water column and the underlying bed. Even when wastewater does not introduce excess sediment into an estuary, it is often measured and modelled in order to quantify the transport of sediment-bound pollutants.

   c) Bacteria and viruses may enter estuaries in runoff from farms and feedlots and in effluent from marinas as well as from municipal or industrial wastewater discharges. These pathogens may be transported to bathing beaches and recreational areas, causing direct human exposure and possibly disease. Pathogens also may be transported to shellfish habitat; there they may accumulate in oysters, clams, and mussels and, subsequently, cause disease when eaten by humans.

   d) Adequate, sustained DO concentrations are a requirement for most aquatic organisms. Seasonal or diurnal depletion of DO, then, disrupts or displaces estuarine communities. Ambient DO levels are affected by many natural processes, such as oxidation of organic material, nitrification, diagenesis of benthic sediments, photosynthesis and respiration by phytoplankton and submerged aquatic vegetation, and reaeration. The natural balance can be disrupted by excessive wastewater loads of organic material, ammonia, and nutrients. Other sources of nutrients, such as runoff from agricultural, residential, and urban lands and atmospheric deposition, can also disrupt the DO balance. Excessive heat input from power plants can aggravate existing problems. Because of its intrinsic importance, and because it is affected by so many natural and man influenced processes, DO is perhaps the best conventional indicator of water quality problems.

e) Adequate concentrations of nitrogen and phosphorus are important in maintaining the natural productivity of estuaries. Excessive nutrient loading, however, can stimulate overproduction of some species of phytoplankton, disrupting the natural communities. Periodic phytoplankton "blooms" can cause widely fluctuating DO concentrations, and DO depletion in benthic and downstream areas. Nutrient loads can be introduced in wastewater and runoff and through atmospheric deposition.

f) High concentrations of ammonia, many organic chemicals, and metals can disable or kill aquatic organisms. Acute toxicity is caused by high exposure to pollutants for short periods of time (less than four days). The toxicity of a chemical can be affected by such environmental factors as pH, temperature, and sediment concentrations. Overall toxicity results from the combined exposure to all chemicals in the effluent and the ambient waters.

g) Lower concentrations of organic chemicals and metals that do not cause aquatic toxicity can be taken up and concentrated in the tissues of estuarine organisms. As fish predators consume contaminated prey, bioaccumulation of these chemicals can occur. This food chain contamination can persist long after the original chemical source is eliminated. Humans that regularly consume tainted fish and shellfish can receive harmful doses of the chemical.

   Human exposure to harmful levels of organic chemicals and metals can also occur through drinking water withdrawals from fresh water tidal rivers.

5.2.15.3.5. Waste Load Allocation Modelling

Because of the wide array of variable elements that must be considered in assessing a receiving water's assimilative capacity, computerized mathematical models are generally employed to make the necessary calculations. In the simplest situations, manual calculations can be performed. In most cases, however, the use of computerized mathematical models will be much more convenient.

5.2.15.3.5.1. Model Selection

The initial step of any waste load allocation study is to define the nature and the extent of the problem. Once this is done, the preferred approach in model selection is to use the simplest model that can be applied to a particular case.

Ideally, the model should include only those phenomena that are operative and important in the receiving water being modeled. The most appropriate procedure for selecting a model is to first define the phenomena that are important for the particular site-specific analysis to be performed. Activities that help to define phenomena that should be incorporated include the following:

1. review of existing data on waste loads, and receiving water quality; and
2. preliminary mass balance calculations using simple models or equations that provide analytical solutions for various load sources (combined sewer overflows, non-point sources, sediment) and reaction phenomena.

It is also desirable to attempt to anticipate the technical issues with respect to control actions (level of treatment, alternate discharge locations, etc.) and determine whether this will influence the types of reactions that will be important. From the foregoing, the analyst will generally be able to establish the phenomena that should be included in the selected model and the time and space scale of the analysis, which is most appropriate.

Under ideal circumstances, one would select a formal model or analysis approach that included all the phenomena determined to be important in the study area, and which excluded those reactions that are insignificant in the case in question. While this guidance should be followed as much as possible, in practice a calculation framework or model may be selected because it is available or familiar to the analyst.

In such cases, two criteria are important to apply. First, the model selected must be capable of handling all of the important site-specific phenomena considering the time and space scale of the analysis and using the equations and formulations specified. Secondly, provision should be made, where possible, to eliminate from the calculation framework the effect of any phenomena that are insignificant in the site-specific analysis. In some cases, inclusion of phenomena judged to be unimportant on a site-specific basis can increase the level of uncertainty of the analysis and thus directly affect decisions. In these situations, additional data collection, sensitivity runs, and other aspects of the overall waste load allocation program must be considered, in order that phenomena contained in the calculations are adequately addressed.

Additional evaluation criteria for model selection include completeness of computer program documentation, costs for manpower, and computer time.

5.2.15.3.5.2. Model Selection Guidelines

Guidelines for selection of a model fall under two categories: technical and operational. The technical guidelines ultimately are concerned with matching the model capabilities to the important physical and biochemical processes of the prototypical system. The operational guidelines are concerned with the ease and cost associated with model operation.

The following is the sequence of model selection guidelines (both technical and operational), with a brief discussion of the considerations involved:

1. Technical Guideline #1 - Determine Important Features of the Prototypical System that are Required in the Analysis

   Site-specific data should be collected and reviewed to understand the system and establish the important factors associated with the identified problem. Valuable information can also be obtained from other experienced professionals, especially those who have modelling experience or site-specific field experience, and from personal site visits.

2. Technical Guideline #2 - Review Available Models and Model Capabilities

   There are a wide number of models available capable of performing waste load allocations. It is important to be aware of those capabilities that involve a substantial increase in complexity.

3. Technical Guideline #3 - Match Important Features of the Prototypical System with Model Capabilities - An important step in model selection is comparing the important features of the prototypical system with the model capabilities and selecting, as technically acceptable, those models whose capabilities match the features of the system. A rule of thumb is to select the simplest model(s) that retains all-important features in the prototypical system. Choosing a more complex model is not cost effective since data requirements and computer cost tend to increase rapidly. An overly complex program will not usually result in an improved simulation and may increase uncertainty in the analysis.

4. Technical Guideline #4 - Confirm Selection of Technically Acceptable Models

   To confirm that the models are indeed technically appropriate, the potential user should consult the user's manual and other support documents, contact and discuss the potential application with the DOEC, and consult with other experienced professionals.

5. Operational Guideline #1 - Selection of Candidate Models Based on Ease of Application

   Once a technically acceptable model has been selected, it is necessary to estimate the ease of applying it. However, it is very difficult to evaluate the adequacy of documentation and support and realistically estimate costs without prior experience with the model. Therefore, it is recommended that the DOEC be consulted. It may be possible that special support arrangements (including short courses or informational or personnel exchanges) are available under existing agreements or otherwise could be made available to the potential user. The DOEC may also be able to provide the potential user with a list of local users who could be contacted for information regarding their past or current experience with the computer program associated with the model.

6. Operational Guideline #2 - Selection of Candidate Models Based on Cost of Application and Problem Significance

   It is difficult to estimate overall costs involved in a model application because each application differs in scope and complexity, and the ability to solve or avoid certain problems is very dependent on the experience and technical background of the analysts involved. However, machine requirements and costs associated with typical runs are usually estimated in the program documentation. As a rule, the simpler the model, the less expensive it is to apply. Again, it is essential that the DOEC and other experienced professionals be contacted for information or assistance.

   Once an estimate of the costs of application has been made, it should be compared with the benefits of using the program as part of the water quality modelling effort and the overall importance of the problem. In other words, the WLA study costs should be consistent with the economic, social, or environmental values associated with the problem and its solution.

7. Operational Guideline #3 - Selection of Candidate Models Based on Data Availability and Data Acquisition Costs

   All models require data for input, calibration, and verification. It is best if model selection is not restricted by availability of data and the decision is made to acquire the specific type of data required for the model. On the other hand, if data availability is a constraint, selection of a less sophisticated model than would be warranted on technical grounds may be appropriate.

The first step in model selection is to determine which programs are technically acceptable, based on an understanding of the important physical and biochemical processes in the prototypical system. The second step is to determine the ease and costs of application of those that are technically acceptable. The result of the second step is a list of candidate models that may or may not be ranked according to convenience and cost. The final selection of the preferred model from the list of candidates is based on the overall judgment of the potential user taking into account all of the factors discussed.

5.2.15.3.5.3. Modelling Procedures

The typical development of a site-specific water quality model follows the following three main steps:

1. Initial Assessment - The historical data are reviewed and employed in conjunction with initial model runs, which compare calculated and observed water quality to:
   a) Confirm existing or future water quality problems;
   b) Define the loads, sources, and sinks that control water quality;
   c) Define the important reactions that control water quality; and
   d) Define issues in the area of transport that must be resolved.

   The initial assessment is the first step of the process aimed at understanding the factors controlling water quality. The initial assessment activity is a first full step in understanding quantitatively the factors controlling water quality. It is not a preliminary analysis; instead, the initial understanding is translated into a field and experimental program whose data output begins to challenge and strengthen the understanding of the system.

2. Field Program - This task translates the results of the initial assessment into a practical field program that can be carried out on the receiving water and in the laboratory using the resources and manpower required and/or available.

3. Model Calibration - Following the selection of an appropriate model and the collection of the relevant field data, it is necessary to calibrate the model. Model calibration is necessary because of the semi-empirical nature of present day water quality models. In model calibration activities, the data from the field program are employed to define model criteria, constants and equations. Water quality calculations using the model are developed for the various conditions associated with each of the water quality data sets. These conditions

include those associated with the historical data and the data collected in the field program. Adjustments in the value of model parameters must be made in a consistent fashion for all conditions.

The results of these activities are a set of consistent model parameters, which are then employed to develop water quality calculations for the conditions associated with all available data sets. Comparisons of calculated and observed water quality profiles should be developed. The model runs and calculations employed to search for and define the series of consistent coefficients should be retained since they can provide an indication of system sensitivity.

5.2.15.4. Registration Under the Environmental Assessment Regulations

A trunk sewer pipeline outfall (Sewage/Wastewater Outfall) with 200 mm inside diameter or greater and with a tributary population of 1000 or more or equivalent flow shall be registered.

5.3. Wastewater Pumping Stations

5.3.1. Energy Requirements

In view of the rising costs of energy and the possibility of future energy shortages, designers should attempt to minimize the number of wastewater pumping stations required in sewage collections systems. Deeper gravity sewers, inverted siphons, or aerial sewers may not only eliminate the need for pumping stations, but may prove to be economically more attractive in the long term and should be examined.

5.3.2. Flooding

Wastewater pumping station structures, electrical and mechanical equipment shall be protected from physical damage during major floods and remain fully operational.

5.3.3. Accessibility and Security

The pumping station shall be readily accessible by maintenance vehicles during all weather conditions. The facility should be located off the traffic way of streets and alleys. It is recommended that security fencing and access hatches with locks be provided.

5.3.4. Grit

Where it is necessary to pump wastewater prior to grit removal, the design of the wet well and pump station piping shall receive special consideration to avoid operational problems from the accumulation of grit.

5.3.5. Safety

Adequate provision shall be made to effectively protect maintenance personnel from hazards. Equipment for confined space entry in accordance with Occupational Health and Safety requirements shall be provided for all wastewater pumping stations.

5.3.6. Design

The following sub-sections should be given consideration in the design of wastewater pumping stations.

5.3.6.1. Type

Wastewater pumping stations in general use, fall into four types: wet well/dry well, submersible, suction lift and screw pump, however as pumping and power requirements increase wet well/dry well type stations are preferred. The typical efficiencies of the two types of pumping systems, along with capital, operation and maintenance costs should be considered when choosing between the two types of stations.

5.3.6.2. Access and Safety Landings

Suitable and safe means of access shall be provided to dry wells and to wet wells. Access to wet wells containing either bar screens or mechanical equipment requiring inspection or maintenance shall conform to Section 5.5.1.2. Removable ladders may be provided in small stations where it is impractical to install stairways.

Safety landings shall be installed in all manholes and pumping stations as per the Department of Municipal and Provincial Affairs, *Municipal Water, Sewer and Roads Specifications*.

5.3.6.3. Buoyancy

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

5.3.6.4. Construction Materials

Materials shall be selected that are appropriate under conditions of exposure to hydrogen sulphide and other corrosive gases, greases, oils and other constituents frequently present in wastewater. This is particularly important in the selection of metals and paints. Contact between dissimilar metals should be avoided or other provisions made to minimize galvanic action.

5.3.6.5. Separation

Dry wells including their superstructure shall be completely separated from the wet well. Common walls must be gas tight.

5.3.6.6. Equipment Removal

Structures and equipment shall be constructed so that there will be convenient facilities to remove pumps and motors. Large pumps shall be accessible to track cranes or suitable portable block and tackle equipment.

5.3.6.7. Dry Space

Dry space should be provided, sufficient in size to accommodate the pumps, motors, valves, piping, etc., without congestion for operation or repairs and maintenance. The floor should slope to a sump equipped for positive drainage.

5.3.7. Pumps

5.3.7.1. Number of Pumps

Each pumping station shall be provided with at least two pumps, and in large main pumping stations, three or more pumps shall be provided.

5.3.7.2. Type of Pumps

The pumps shall be designed specially for pumping sewage and should be set below the high water level of the wet well in order to be self-priming. Otherwise, the pump should be so placed that under normal operating conditions, it will operate under a positive suction pressure.

5.3.7.3. Pump Capacities

If only two units are provided, they shall have the same capacity. Each shall be capable of handling flows in excess of the expected maximum flow. In larger pumping stations where three or more pumps are provided, they should be designed to fit actual flow conditions and shall be of such capacity that, with the largest unit out of service, the remaining units can pump the maximum flow of sewage. Where discharge is directly into the treatment plant, pump capacities should be so selected that one pump will operate almost constantly. The other pumping units shall automatically go into operation whenever the flow exceeds the capacity of the first pump. The pumps should be on a preset cycling arrangement so that one pump is not always the duty pump.

5.3.7.4. Pumping Rates

The pumps and controls of main pumping stations and especially those operated as part of a treatment works, should be selected to operate at varying delivery rates which will permit discharging sewage from the station to the treatment works at approximately its rate of delivery to the pump station. The station design capacity shall be based on peak hourly flow and should be adequate to maintain a minimum velocity of 0.6 m/s in the force main.

5.3.7.5. Pump Connections

The pump suction and discharge lines shall be equipped with suitable check and gate valves to facilitate repairs and shall in no case be less than 100 mm in diameter. Full closing valves shall be installed on the suction and discharge piping to each pump, and a check valve shall be installed in the discharge line of the pump. Where space will not permit the installation of all types of valves, the connecting lines shall be equipped with indicators to show their open and closed positions. Potable water lines shall not be directly connected to any sewage pump, suction or discharge line, for priming, flushing, lubricating, or for any other purpose.

5.3.7.6. Pump Openings

Pumps handling raw wastewater shall be capable of passing spheres of at least 75 mm in diameter.

5.3.7.7. Dry Well Dewatering

A sump pump equipped with dual check valves shall be provided in the dry well to remove leakage or drainage with discharge above the maximum high water level of the wet well. Water ejectors connected to a potable water supply will not be approved. All floor and walkway surfaces should have an adequate slope to a point of drainage. Pump seal leakage shall be piped or channelled directly to the sump. The sump pump shall be sized to remove the maximum pump seal water discharge, which would occur in the event of a pump seal failure.

5.3.7.8. Electrical Equipment

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

5.3.7.9. Intake

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

5.3.7.10. Controls

Control float tubes and bubbler lines should be located, as not to be unduly affected by turbulent flows entering the well or by the turbulent suction of the pumps. Provision shall be made to automatically alternate the pumps in use.

5.3.8. Piping and Valves

5.3.8.1. Piping

Flanged pipe and fittings shall be used for exposed piping inside the pump stations. A flexible connection shall be installed in the piping to each pump so that the pump may be removed easily for repairs. A semi-flexible wall sleeve is recommended where a pipe passes through a wall of the station.

5.3.8.2. Suction Line

Suitable shutoff valves shall be placed on the suction line of dry pit pumps.

5.3.8.3. Discharge Line

Suitable shutoff and check valves shall be placed on the discharge line of each pump. The check valve shall be located between the shutoff valve and the pump. Check valves shall be suitable for the material being handled and shall be placed on the horizontal portion of discharge piping except for ball checks, which may be placed in the vertical run. Valves shall be capable of withstanding normal pressure and water hammer.

All shutoff and check valves shall be operable from the floor level and accessible for maintenance. Outside levers are recommended on swing check valves.

5.3.9. Wet Wells

5.3.9.1. Divided Wells

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

5.3.9.2. Size

The effective capacity of the wet well below the inlet sewer, where possible, shall provide a holding period of between 10 and 30 minutes for the average flow. To avoid septicity problems wet wells should not provide excessive retention times. Wet wells should be of minimum size consistent with pump capacities and not be designed to provide storage in cases of power failure. Alternatively, the retention period of a wet well may be 5 to 10 minutes when variable speed pumping is employed, and when the number of pumps is adequate.

Wet well sizing will be influenced by factors such as the volume required for pump cycling; dimensional requirements to avoid turbulence problems; the vertical separation between pump control points; the inlet sewer elevation(s); capacity required between alarm levels and basement flooding and/or overflow elevation; number of and horizontal spacing between pumps.

To minimize pumping costs and wet well depth, normal high water level (pump start elevation) may be permitted to be above the invert of the inlet sewer(s) provided basement flooding and/or solids deposition will not occur. Where these problems cannot be avoided, the high water level (pump start elevation) should be approximately 300 mm below the invert of the inlet sewer. Low water level (pump shut down) should be at least 300 mm or twice the pump suction diameter above the centre lien of the pump volute. The bottom of the wet well should be no more than D/2, nor less than D/3 below the mouth of the flared intake elbow.

5.3.9.3. Floor Slope

The bottom of the wet well should slope sharply to the suction pipe of the pump to minimize the accumulation of sewage solids. The horizontal area of the hopper bottom shall be no greater than necessary for proper installation and function of the inlet. Slopes of 1.0 vertical to 1.0 horizontal are required.

5.3.9.4. Hazardous Area

Wet wells are classified as Hazardous Location and the requirements of the Canadian Electrical Code must be satisfied for all electrical installations in wet wells, or the equipment must be CSA approved for use in sewage-wet wells.

5.3.9.5. Air Displacement

Covered wet wells shall have provisions for air displacement to the atmosphere, such as an inverted "J" tube or other means.

5.3.10. Safety Ventilation

Ventilation shall be provided for all pumping stations, including both wet and dry wells. Mechanical ventilation shall be provided for all dry wells below the ground surface, and for wet wells where equipment is located. There shall be no interconnection between the wet well and dry well ventilation systems.

5.3.10.1. Air Inlets and Outlets

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

5.3.10.2. Electrical Controls

Switches for operation equipment should be marked and located conveniently. All intermittently operated ventilation equipment shall be interconnected with the respective pit lighting system. Consideration should be given also to automatic controls where intermittent operation is used. The manual lighting/ventilation switch shall override the automatic controls. For a two speed ventilation system with automatic switch over where gas detection equipment is installed, consideration should be given to increasing the ventilation rate automatically in response to the detection of hazardous concentrations of gases or vapours.

5.3.10.3. Fans, Heating and Dehumidification

The fan wheel shall be fabricated from non-sparkling material. Automatic heating and dehumidification equipment shall be provided in all dry wells. The electrical equipment and components shall meet the requirements of Section 5.3.7.8.

5.3.10.4. Wet Wells

Wet well ventilation may be either continuous or intermittent. Ventilation, if continuous shall provide at least 12 complete air changes per hour. Air shall be forced into the wet well by mechanical means rather than solely exhausted from the wet well. The air change requirements shall be based on 100 percent fresh air. Portable ventilation equipment shall be provided for use at submersible pump stations and wet wells with no permanently installed ventilation equipment.

5.3.10.5. Dry Wells

Dry well ventilation may be either continuous or intermittent. Ventilation if continuous, shall provide at least 6 complete air changes per hour; if intermittent, at least 30 complete air changes per hour. A system of two-speed ventilation with an initial ventilation rate of 30 changes per hour for 10 minutes and automatic switch over to 6 changes per hour may be used to conserve heat. The air change requirements shall be based on 100 % fresh air.

5.3.11. Flow Measurement

Suitable devices for measuring flow shall be provided at all pumping stations. Flow control mechanisms shall be located so that flow currents created by the entering sewage or by pump suction will not adversely affect them. Provision shall be made to prevent floating material in the wet well from interfering with the operation of the controls. When a float tube is installed in the dry well, its height shall be sufficient so as to prevent overflow of sewage into the dry well. At larger pumping stations, consideration should be given to installing suitable devices for measuring sewage flows and power consumption. Indicating, totalizing and recording flow measurement shall be provided at pumping stations with a 75 L/s or greater design peak flow. Elapsed time meters used in conjunction with annual pumping rate tests may be acceptable for pump stations with a design peak hourly flow up to 75 L/s provided sufficient metering is configured to measure the duration of individual and simultaneous pump operation. These instruments shall be capable of being integrated (electrically) with automatic samplers.

5.3.12. Potable Water Supplies

There shall be no physical connection between any potable water supply and a wastewater pumping station, which under any conditions might cause contamination of the potable water supply.

5.3.13. Suction-Lift Pump Stations

Suction-lift pumps shall meet the requirements of Section 5.3.6.

5.3.13.1. Pump Priming and Lift Requirements

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

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

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

5.3.13.2. Equipment, Wet Well Access, and Valving Location

The pump equipment compartment shall be above the grade or offset and shall be effectively isolated from the wet well to prevent a hazardous and corrosive sewer atmosphere from entering the equipment compartment. Wet well access shall not be through the equipment compartment and shall be at least 580 mm in diameter. Gasketed replacement plates shall be provided to cover the opening to the wet well for pump units removed for servicing. Valving shall not be located in the wet well.

5.3.14. Submersible Pump Stations

Submersible pump stations shall meet the requirements as outlined in the previous sections, except as modified below.

5.3.14.1. Construction

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

5.3.14.2. Pump Removal

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

5.3.14.3. Electrical Equipment

5.3.14.3.1. Power Supply and Control Circuitry

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

5.3.14.3.2. Controls

The motor control centre shall be located outside the wet well, be readily accessible, and be protected by a conduit seal or other appropriate measures meeting the requirements of the Canadian Electrical Code, to prevent the atmosphere of the wet well from gaining access to the control centre. The seal shall be so located that the motor may be removed and electrically disconnected without disturbing the seal. When such equipment is exposed to weather, it shall meet the requirements of weatherproof equipment.

5.3.14.3.3. Power Cord

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

5.3.14.4. Valves

Check valves that are integral to the pump need not be located in a separate valve pit provided that the valve can be removed from the wet well in accordance with Section 5.3.14.2.

5.3.15. Alarm Systems

Alarm systems shall be provided for pumping stations. The alarm shall be activated in cases of power failure, sump pump failure, pump failure, unauthorized entry, or any cause of pump station malfunction. Pumping station alarms shall be telemetered to a municipal facility that is manned 24 hours a day. If such a facility is not available and a 24-hour holding capacity is not provided, the alarm shall be telemetered to municipal offices during normal working hours and to the home of the responsible person(s) in charge of the lift station during off-duty hours. Audio-visual alarm systems with a self contained power supply may be acceptable in some cases in lieu of the telemetering system outlined above, depending upon location, station holding capacity and inspection frequency.

5.3.16. Emergency Operation

Pumping stations and collection systems shall be designed to prevent or minimize bypassing of raw sewage. For use during periods of extensive power outages, mandatory power reductions, or uncontrolled storm events, consideration should be given to providing a controlled, high-level wet well overflow to supplement alarm systems and emergency power generation in order to prevent backup of sewage into basements, or other discharges which may cause severe adverse impacts on public interests, including public health and property damage. Where a high level overflow is utilized, consideration shall also be given to the installation of storage/detention tanks, or basins, which shall be made to drain to the station wet well. Storage capacity should be related to frequency and length of power outages for the area. Power outage history can be obtained from the power supply company for the grid where the pump station is to be located. This data can then be utilized with peak design flows for design of storage facilities to minimize overflow. Where elimination of overflows is not practical this data can be used to predict frequency and quantities of overflows. Where public water supplies, shellfish production, or waters used for culinary or food-processing purposes exist, overflows shall not be permitted.

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

1. Storage capacity, including trunk sewers, for retention of wet weather flows (storage basins must be designed to drain back into the wet well or collection system after the flow recedes); and

2. An in-place or portable pump, driven by an integral combustion engine meeting the requirements of Section 5.3.17, capable of pumping from the wet well to the discharge side of the station.

5.3.16.2. Overflows
Overflows are not permitted unless special written permission has been obtained from the DOEC.

5.3.17. Equipment Requirements

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

5.3.17.1.1. Engine Protection
The engine must be protected from operating conditions that would result in damage to equipment. Unless continuous manual supervision is planned, protective equipment shall be capable of shutting down the engine and activating an alarm on-site and as provided in Section 5.3.15. Protective equipment shall monitor for conditions of low oil pressure and overheating, except that oil pressure monitoring will not be required for engines with splash lubrication.

5.3.17.1.2. Size
The engine shall have adequate rated power to start and continuously operate all connected loads.

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

5.3.17.1.4. Engine Ventilation
The engine shall be located above grade with adequate ventilation of fuel vapours and exhaust gases.

5.3.17.1.5. Routine Start-up
All emergency equipment shall be provided with instructions indicating the need for regular starting and running of such units at full loads.

5.3.17.1.6. Protection of Equipment

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

5.3.17.2. Engine-Driven Pumping Equipment

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

5.3.17.2.1. Pumping Capacity

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

5.3.17.2.2. Operation

The engine and pump shall be equipped to provide automatic start-up and operation of pumping equipment. Provisions shall also be made for manual start-up. Where manual start-up and operation is justified, storage capacity and alarm systems must meet the requirements of Section 5.3.17.2.3.

5.3.17.2.3. Portable Pumping Equipment

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

5.3.17.3. Engine-Driven Generating Equipment

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

5.3.17.3.1. Generating Capacity

Generating unit size shall be adequate to provide power for pump motor starting current and for lighting, ventilation and other auxiliary equipment necessary for safety and proper operation of the pumping station. The operation of only one pump during periods of auxiliary power supply must be justified. Such justification may be made on the basis of maximum anticipated flows relative to single-pump capacity, anticipated length of power outage and storage capacity. Special sequencing controls shall be provided to start pump motors unless the generating equipment has capacity to start all pumps simultaneously with auxiliary equipment operating.

5.3.17.3.2. Operation

Provisions shall be made for automatic and manual start-up and load transfer. The generator must be protected from operating conditions that would result in damage to equipment. Provisions should be considered to allow the engine to start and stabilize at operating speed before assuming the load. Where manual start-up and transfer is justified, storage capacity and alarm systems must meet requirements of Section 5.3.17.2.3.

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

5.3.17.4. Independent Utility Substations
Where independent substations are used for emergency power, each separate substation and its associated transmission lines shall be capable of starting and operating the pump station at its rated capacity.

5.3.18. Instructions and Equipment
The operating authority of sewage pumping stations shall be supplied with a complete set of operational instructions including; emergency procedures, maintenance schedules, tools and such spare parts as may be necessary.

5.3.19. Safety and Housekeeping
Maximum consideration shall be given to providing safe working conditions for the operators; this means, among other things that provision for a high standard of housekeeping shall be essential. Poor housekeeping and poor safety practices in sewage pumping stations cannot be tolerated.

For example, portable ladders, preferably ships' ladders, shall be used rather than manhole ladder rungs. Ships' ladders, stairs with non-slip treads, or mechanical lifts shall be provided for all installations. All electrical wiring shall be properly grounded; lighting shall be adequate in all locations, etc.

5.3.20. Forcemains

5.3.20.1. Diameter and Velocity
In general, forcemains shall be a minimum of 100 mm in diameter. Velocities should be in the range of 0.6 to 2.5 m/s.

5.3.20.2. Termination
Forcemains shall enter the gravity sewer at a point not more than 600 mm above the flow line of the receiving manhole. A 45° bend may be considered to direct the flow downwards.

5.3.20.3. Design Friction Losses

5.3.20.3.1. Friction Coefficient

Friction losses through forcemains shall be based on the Hazen-Williams formula or other acceptable method. When the Hazen-Williams formula is used, the following values for C shall be used for design:

1. Unlined iron or steel: C = 100; and
2. All other: C = 120

5.3.20.3.2. Maximum Power Requirements

When initially installed, forcemains will have a significantly higher “C” factor. The effect of the higher “C” factor should be considered in calculating maximum power requirements and duty cycle time to prevent damage to the motor.

5.3.20.4. Air and Vacuum Relief Valves

Automatic air relief valves shall be placed at all high points in the forcemain to prevent air locking. Vacuum relief valves may be necessary to relieve negative pressures on force mains. The force main configuration and head conditions should be evaluated as to the need for and placement of vacuum relief valves.

5.3.20.5. Pipe and Design Pressure

The forcemain and fittings, including reaction blocking, shall be designed to withstand normal pressure and pressure surges (water hammer) and associated cyclic reversal of stresses that are expected with the cycling of wastewater lift stations. Surge protection chambers should be evaluated.

5.3.20.6. Separation

Watermains and sewage forcemains are to be installed in separate trenches 3 metres apart. The soil between the trenches shall be undisturbed. Forcemains crossing watermains shall be laid to provide a minimum vertical distance of 450 mm between the outside of the forcemain and the outside of the watermain. The watermain shall be above the forcemain. At crossings, one full length of water pipe shall be located so both joints will be as far from the forcemain as possible. Special structural support for the watermain and the forcemain may be required.

5.3.20.7. Slope and Depth

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

5.3.20.8. Identification

Where forcemains are constructed of material, which might cause the forcemain to be confused with potable watermains, the forcemain should be appropriately identified.

5.3.20.9. Testing

Leakage is defined as the amount of water supplied from a water storage tank in order to maintain test pressure for 2 hours. The allowable leakage is 0.03 L/mm pipe diameter per 300 m, per hour for a working pressure of 1000 kPa. For other working pressures, test in accordance with the latest version of AWWA C600.

5.4. Wastewater Treatment

5.4.1. Plant Location

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

1. Proximity to residential areas;
2. Direction of prevailing winds;
3. Accessibility by all-weather roads;
4. Area available for expansion;
5. Protection against flooding;
6. Local zoning requirements;
7. Local soil characteristics, geology, hydrology, and topography available to minimize pumping (selected elevations to maximize gravity flow through the works);
8. Access to receiving stream;
9. Downstream uses of the receiving water body;
10. Compatibility of treatment process with the present and planned future land use, including noise, potential odours, air quality, and anticipated sludge processing and disposal techniques; and
11. Proximity to surface water supplies and water wells.

Where a site must be used which is critical with respect to these items, appropriate measures shall be taken to minimize adverse impacts.

5.4.1.1. Flood Protection

The treatment plant structures, electrical and mechanical equipment shall be protected from physical damage by the 100-year flood. Treatment plants should remain fully operational and accessible during floods. This requirement applies to new construction and to existing facilities undergoing major modification.

5.4.2. Quality of Effluent

The quality of effluent shall meet the *Environmental Control Water and Sewer Regulations, 2003*.

5.4.3. Design

5.4.3.1. Type of Treatment

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

1. Present and future effluent requirements;
2. Location of and local topography of the plant site;
3. Space available for future plant construction;
4. The effects of industrial wastes likely to be encountered;
5. Ultimate disposal of sludge;
6. System capital costs;
7. System operating and maintenance costs, including basic energy requirements;
8. Process complexity governing operating personnel; and
9. Environmental impact on present and future adjacent land use.

5.4.3.2. Required Engineering Data for New Process and Application Evaluation

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

To determine that such new processes and equipment or applications have a reasonable and substantial chance of success, the DOEC may require the following:

1. Adequate monitoring observations, including test results and engineering evaluations, demonstrating the efficiency of such processes under varied flow regimes;

2. Detailed description of the test methods;

3. Testing, including appropriate composite samples, under various ranges of strength and flow rates (including diurnal variations) and waste temperatures over a sufficient length of time to demonstrate performance under climatic and other conditions, which may be encountered in the area of the proposed installations; or

4. Other appropriate information.

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

5.4.3.3. Design Loads

5.4.3.3.1. Hydraulic Design

***New Systems** - the design for sewage treatment plants to serve new sewage systems shall be based on an average daily flow of 340 L/cap,* unless all season flow data is available, or other justification upon which to better estimate flow is provided.

Existing Systems - where there is an existing system the volume and strength of existing flows shall be determined. The determination shall include both dry-weather and wet-weather conditions. Samples shall be taken and composited so as to be accurately representative of the strength of the wastewater. The following information shall be recorded where appropriate for the design of the works:

1. Peak rates of flow over a sufficient period of time, which would adversely affect the detention time of treatment units or the flow characteristics of the conduits;
2. Percentage of industrial waste flows; and
3. Wet-weather peak flows.

Flow Equalization - facilities for the equalization of flows and organic shock load shall be considered at all plants, which are critically affected by surge loadings.

5.4.3.3.2. Organic Design

Domestic waste treatment design shall be on the basis of at least 0.08 kg of BOD per capita per day and 0.09 kg of suspended solids per capita per day, unless other information is submitted to justify alternate designs.

Domestic waste treatment plants that will receive commercial, institutional and industrial wastewater flows shall be designed to include the waste loads as per Table 5.1.

Septage and leachate may contribute significant organic load and other materials, which can cause operational problems. If septage and/or leachate are to be discharged to the wastewater treatment facility, it must be approved by the DOEC.

When an existing treatment works is to be upgraded or expanded, the organic design shall be based upon the actual strength of the wastewater, with an appropriate increment for growth.

5.4.3.3.3. Minimum Parameters

In the absence of documented data for a specific wastewater treatment facility, the minimum design parameters provided in Sections 5.4.3.3.1 and 5.4.3.3.2 shall be used.

5.4.3.4. Shock Effects

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

5.4.3.5. Design by Analogy

Data from similar municipalities may be utilized in the design of new systems.

5.4.3.6. Conduits and Piping

All piping and channels should be designed to carry the maximum expected flows. The incoming sewer should be designed for free discharge. Bottom corners of the channels must be filleted. Pockets and corners where solids can accumulate should be eliminated. Suitable gates should be placed in channels to seal off unused sections, which might accumulate solids. The use of shear gates or stop planks is permitted where they can be used in place of gate valves or sluice gates. Non-corrodible materials shall be used for these control gates. Pipe and accessories used for conduits shall conform to the standard specifications of the AWWA or equivalent. In general, the following factors shall be considered in designing conduits: carrying capacity, maximum pressures, present and future water hammer, hydraulic grade, traffic loads, laying conditions, expansion and contraction, anchorage at bends and joints, and depth of cover.

5.4.3.7. Arrangement of Units

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

5.4.3.8. Flow Division Control

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

5.4.4. Plant Details

5.4.4.1. Installation and Inspection of Mechanical Equipment

The specifications should be so written that the manufacturer would commission the installation and initial operation of major items of mechanical equipment, and provide appropriate training to the operator.

5.4.4.2. Unit Dewatering, Flotation Protection and Plugging

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

5.4.4.3. Unit Bypasses

5.4.4.3.1. Removal from Service

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

5.4.4.3.2. Unit Bypass During Construction
A plan for the method and level of treatment to be achieved during construction shall be developed and included in the facility plan that must be submitted to the DOEC for review and approval. This approved treatment plan must be implemented by inclusion in the plans and specifications to be bid for the project.

5.4.4.4. Construction Materials
Due consideration should be given to the selection of materials to be used in sewage treatment works, because of the possible presence of hydrogen sulphide and other corrosive gases, greases, oils and similar constituents frequently present in sewage. This is particularly important in the selection of metals and paints. Contact between dissimilar metals should be avoided to minimize galvanic action.

5.4.4.5. Painting
In order to facilitate identification of piping, particularly in large plants, it is suggested that the different lines have contrasting colours. The colour code, as per Table 5.5, is recommended for purposes of standardization for all sewage systems in the province:

Table 5.5
Paint Colour Code for Sewage Systems

Contents	Colour Scheme
Raw Sludge line	Brown w/ black bands
Sludge Recirculation suction line	Brown w/ yellow bands
Sludge draw off line	Brown w/ orange bands
Sludge Recirculation discharge line	Brown
Sludge gas line	Orange(or red)
Natural Gas line	Orange(or red) w/ black bands
Potable water line	Blue
Non-potable water system	Blue with Black bands
Chlorine line	Yellow
Sulphur Dioxide	Yellow w/ red bands
Sewage (wastewater) line	Grey
Compressed air line	Green
Water lines for heating digesters or buildings	Blue w/ 150mm red band spaced 750mm apart

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

5.4.4.6. Operating Equipment
The specifications should include a complete outfit of tools, accessories for the plant operator's use, such as squeegees, wrenches, valve keys, rakes, shovels, spare parts, etc. A portable pump is desirable. Readily accessible storage space and workbench facilities shall be provided and consideration shall be given to provision of a garage for large equipment storage, maintenance and repair.

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

5.4.4.8. Erosion Control During Construction
Effective site erosion control shall be provided during construction.

5.4.5. Plant Outfalls
Refer to Section 5.2.15.

5.4.6. Essential Facilities

5.4.6.1. Emergency Power Facilities
A standby source of power shall be provided where the discharge of raw or partially treated sewage may endanger public health or cause damage to the environment and where the treatment process or biomass may be adversely affected due to oxygen depletion and septic conditions. The need for standby power and the extent of equipment requiring operation by standby power must be individually assessed for each sewage treatment plant. Some of the factors which will require consideration in making the decisions regarding standby power and the processes to be operated by the standby power equipment are as follows:

1. Reliability of primary power source;

2. Number of power feeder lines supplying grid system, number of alternate routes within the grid system, and the number of alternate transformers through which power could be directed to the sewage treatment plant;

3. Whether sewage enters the plant by gravity or is pumped;

4. Type of treatment provided;

5. Pieces of equipment which may become damaged or overloaded following prolonged power failure;

6. Assimilation capacity of the receiving waters and ability to withstand higher pollution loadings over short time periods; and

7. Other uses of the receiving water.

Each specific installation should provide for the following considerations:
1. Means for illuminating working areas to ensure safe working conditions; and

2. Standby power source or equivalent to power pumps, motorized valves and control panels that are necessary to maintain the sewage flow through the treatment plant.

5.4.6.1.1. Power for Aeration
Standby generating capacity normally is not required for aeration equipment used in the activated sludge process. In cases where a history of long-term (4 hours or more) power outages has occurred, auxiliary power for minimum aeration of the activated sludge will be required. Full power generating capacity may be required by the reviewing authority on certain critical stream segments.

5.4.6.1.2. Power for Disinfection
Continuous disinfection, where required, shall be provided during all power outages. Continuous dechlorination is required for all systems using chlorine.

5.4.6.2. Measurement of Flow
5.4.6.2.1. Facilities
Flow measurement facilities shall be provided at all plants. Indicating, totalizing and recording flow measurement devices shall be provided for all treatment plants. All flow measurement equipment must be sized to function effectively over the full range of flows expected and shall be protected against freezing.

5.4.6.2.2. Location
Consideration shall be given to the provision of automatic samplers at large plants.

Flow measurement facilities shall be provided to measure the following flows:
1. Plant influent or effluent flow;
2. Plant influent flow, if influent flow is significantly different from effluent flow, both shall be measured. This would apply for installations such as lagoons, sequencing batch reactors and plants with excess flow storage or flow equalization;
3. Excess flow treatment facility discharges;
4. Other flows required to be monitored under the provisions of the Permit to Operate; and
5. Other flows such as return activated sludge, waste activated sludge, recirculation and recycle required for plant operational control.

5.4.6.2.3. Hydraulic Conditions
Flow measurement equipment including approach and discharge conduit configuration and critical control elevations shall be designed to ensure that the required hydraulic conditions necessary for accurate measurement are provided. Conditions that must be avoided include turbulence, eddy currents, and air entrainment

5.4.6.3. Septicity
Where it can be expected that the raw sewage will be largely devoid of dissolved oxygen, provisions should be made for aeration, recirculation or other means of offsetting septic action and odour nuisance and to ensure efficient operation of the plant.

5.4.6.4. Water Supply
An adequate supply of potable water, under pressure shall be provided for sanitary and drinking purposes, use in the laboratory and for general cleanliness around the plant. All plumbing shall comply with the requirements of the Canadian Plumbing Code. No piping or other connections shall exist in any part of the treatment works, which, under any conditions, might cause the contamination of a potable water supply. The chemical quality should be checked for suitability for its intended use such as in heat exchangers, chlorinators, etc.

The municipal water system and the potable water piping system within the treatment building shall be protected from the potential backflow of sewage due to back-siphonage or backpressure with the use of certified backflow prevention devices. In this case, the minimum device(s) to be used on the building water service line(s) shall be a reduced pressure zone backflow preventer, meeting or exceeding CSA 64.4. A certified tester must test this device on an annual basis. For protection of water users within the building, appropriate vacuum breakers, as determined by the Engineer, shall be installed on any threaded water connection. There shall be no connection made to the potable water supply within the treatment building between the reduced pressure zone backflow preventer and the street source.

5.4.6.4.1. Direct Connections
Potable water from a municipal or separate supply may be used directly at points above grade for the following hot and cold supplies:
1. Lavatory;
2. Water closet;
3. Laboratory sink (with vacuum breaker);
4. Shower
5. Drinking fountain;
6. Eye wash fountain; and
7. Safety shower.

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

5.4.6.4.2. Indirect Connections

Where a potable water supply is to be used for any purpose in a plant other than those listed in Section 5.4.6.4.1, a break tank, pressure pump and pressure tank shall be provided. Water shall be discharged to the tank through an air gap, at least 150 mm above the maximum flood line or the spill line of the tank whichever is higher. A sign shall be permanently posted at every hose bibb, faucet, hydrant or sill cock located on the water system beyond the break tank to indicate that the water is not safe for drinking.

5.4.6.4.3. Separate Potable Water Supply

Where it is not possible to provide water from a public water supply, a separate well may be provided. Location and construction of the well should comply with requirements and regulations of the DOEC. Requirements governing the use of the supply are those contained in Sections 5.4.6.4.1 and 5.4.6.4.2.

5.4.6.4.4. Separate Non-potable Water Supply

Where a separate non-potable water supply is to be provided, a break tank will not be necessary, but all system outlets shall be posted with a permanent sign indicating the water is not safe for drinking.

5.4.6.5. Sanitary Facilities

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

Floor surfaces shall be sloped adequately to a point of drainage.

5.4.6.6. Stairways

Stairways shall be installed in compliance with the National Building Code.

5.4.6.7. Instrumentation and Control

Refer to Section 6.0 on Instrumentation and Control.

5.4.6.8. Laboratory Facilities

Laboratory facilities, suitable for controlling the operations and determining the efficiency of the various treatment units, shall be provided at every plant. A list of laboratory supplies and equipment shall be included in the specifications or a special arrangement such as a cash allowance to cover the specific laboratory equipment needed. The plans shall include sufficient laboratory space and appurtenances in the control building or elsewhere. Specific requirements should be obtained from the DOEC.

5.4.7. Safety

Adequate provision shall be made in accordance with the Occupational Health and Safety requirements to effectively protect the operator and visitors from hazards. The following shall be provided to fulfil the particular needs of each plant:

1. Enclosure of the plant site with a fence and signs designed to discourage the entrance of unauthorized persons and animals;

2. Hand rails and guards around tanks, trenches, pits, stairwells, and other hazardous structures with the tops of walls less than 1 m above the surrounding ground level;

3. Gratings over appropriate areas of treatment units where access for maintenance is required;

4. First aid equipment;

5. "No Smoking" signs in hazardous areas;

6. Protective clothing and equipment, such as self-contained breathing apparatus, gas detection and oxygen depletion equipment, goggles, gloves, hard hats, safety harnesses, eye wash stations, stations, etc.;

7. Potable blower and sufficient hose;

8. Portable lighting equipment complying with the National and Provincial Electrical Code requirements;

9. Gas detectors;

10. Appropriately placed warning signs for slippery areas, non-potable water fixtures, low head clearance areas, open service manholes, hazardous chemical storage areas, flammable fuel storage areas, etc.;

11. Adequate ventilation in pump station areas;

12. Provisions for local lockout on stop motor controls; and

13. Provisions for confined space entry.

In addition, reference should be made to appropriate federal and provincial legislation regarding hazardous chemicals.

5.5. Screening And Grit Removal

5.5.1. Bar Racks and Screens

Coarse bar racks or screens shall be provided as the first treatment stage for the protection of plant equipment against reduced operating efficiency, blockage, or physical damage.

5.5.1.1. Selection Considerations

When considering which types of screening devices should be used, the following factors should be considered:

1. Effect on downstream treatment and sludge disposal operations;
2. Possible damage to comminutor or barminutor devices caused by stones or coarse grit patches;
3. Head losses of the various alternative screening devices;
4. Maintenance requirements;
5. Screenings disposal requirements, and quantities of screenings; and
6. Requirements for a standby unit.

5.5.1.2. Access

Appropriate access provisions should be provided as per appropriate Occupational Health and Safety requirements

5.5.1.3. Ventilation

Appropriate ventilation provisions should be provided as per appropriate Occupational Health and Safety requirements.

5.5.1.4. Design

5.5.1.4.1. Velocity

At the design average rate of flow, the screen chamber should be designed to provide a velocity through the screen of approximately 0.3 m/s to prevent settling, and a maximum velocity during wet weather periods no greater than 0.75 m/s to prevent forcing material through the openings. The velocity shall be calculated from a vertical projection of the screen openings on the cross-sectional area between the invert of the channel and the flow line.

5.5.1.4.2. Bar Spacing

1. Manually Cleaned Screens - Clear openings between bars should be from 25 mm to 45 mm. Design and installation shall be such that they can be conveniently cleaned.

2. Mechanical Screens - Clear openings for mechanically cleaned screens may be as small as 15 mm. Mechanical screens are recommended where the installation is not regularly supervised or where an increase in head results in plant bypass.

5.5.1.4.3. Slope

Manually cleaned screens, except those for emergency use, should be placed on a slope of 30 to 45 degrees with the horizontal.

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

5.5.1.4.5. Materials
Materials for bar screens shall depend on the type of sewage. All components subject to wear, which come in contact with the wastewater, shall be made from 316 stainless steel.

5.5.1.4.6. Lighting
Provide adequate lighting in the screening chamber.

5.5.1.5. Safety
5.5.1.5.1. Railings and Gratings
Manually cleaned screen channels shall be protected by guard railings and deck gratings, with adequate provisions for removal or opening to facilitate raking. Mechanically cleaned screen channels shall also be protected by guard railings and deck gratings. Consideration should also be given to temporary access arrangements to facilitate maintenance and repair.

5.5.1.5.2. Mechanical Devices
Mechanical screening equipment shall have adequate removal enclosures to protect personnel against accidental contact with moving parts and to prevent dripping in multi-level installations. A positive means of locking out each mechanical device and temporary access for use during maintenance shall be provided.

5.5.1.5.3. Drainage
Floor design and drainage shall be provided to prevent slippery areas.

5.5.1.6. Control Systems
5.5.1.6.1. Timing Devices
All mechanical units which are operated by timing devices should be provided with auxiliary controls which will set the cleaning mechanism in operation at preset high water elevation. If the cleaning mechanism fails to lower the high water, a warning should be signalled.

5.5.1.6.2. Electrical Systems and Components
Electrical systems and components (i.e. motors, lights, cables, conduits, switchboxes, control circuits, etc.) in enclosed or partially enclosed spaces where flammable mixtures occasionally may be present (including all space above raw or partially treated wastewater) shall comply with the Canadian Electrical Code, Part 1 and the regulations under the applicable Provincial Power Standards. All electrical components in the headworks room must be explosion proof.

5.5.1.6.3. Manual Override
Automatic controls shall be supplemented by a manual override.

5.5.1.7. Screenings Removal and Disposal

Adequate means for removing screenings shall be provided. Screw conveyor equipment may be necessary depending on the depth of pit and amount of screenings or equipment to be lifted.

Facilities must be provided for handling, storage, and disposal of screenings in a manner acceptable to the DOEC.

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

5.5.1.8. Auxiliary Screens

Where mechanically operated screening or comminuting devices are used, auxiliary manually cleaned screens shall be provided. Where two or more mechanically cleaned screens are used, the design shall provide for taking any unit out of service without sacrificing the capability to handle the peak design flow.

5.5.1.9. Fine Screens

Fine screens may be used in lieu of primary sedimentation providing that subsequent treatment units are designed on the basis of anticipated screen performance. Fine screens are not to be considered equivalent to primary sedimentation

5.5.2. Comminutors

Comminutors or grinders shall be used in plants that do not have primary sedimentation or fine screens and should be provided in cases where mechanically cleaned bar screens will not be used.

5.5.2.1. Location

Where possible comminutors should be located downstream of any grit removal equipment and be protected by a coarse screening device. Provisions for location shall be in accordance with those for screening devices.

5.5.2.2. Size

The comminutor shall be sized to handle design peak hourly flow.

5.5.2.3. Installation

1. A screened bypass channel shall be provided. The use of the bypass channel should be automatic at depths of flow exceeding the design capacity of the comminutor.

2. A 150 mm deep gravel trap should protect each comminutor that is not preceded by grit removal equipment.

3. Gates shall be provided in accordance with Section 5.5.1.4.4.

4. The loss of pressure through comminutors should be minimized.

5.5.2.4. Servicing

Provision shall be made to facilitate servicing units in place and removing units from their location for servicing.

5.5.2.5. Electrical Controls and Motors

Electrical equipment in comminutor chambers where hazardous gases may accumulate shall comply with the Canadian Electrical Code and applicable Provincial Power Standards. Motors in areas not governed by this requirement may need protection against accidental submergence.

5.5.3. Grit Removal Facilities

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

Grit removal facilities should be provided for all sewage treatment plants and are required for plants receiving sewage from combined sewers or from sewer systems receiving substantial amounts of grit. If a plant, serving a separate sewer system, is designed without grit facilities, the design shall include provisions for future installation. Consideration shall be given to possible damaging effects on pumps, comminutors and other preceding equipment and the need for additional storage capacity in treatment units where grit is likely to accumulate.

5.5.3.1. Location

Grit removal facilities should be located ahead of pumps and comminuting devices. Coarse bar racks should be placed ahead of grit removal facilities.

5.5.3.2. Accessibility

Consideration should be given in the design of grit chambers to provide safe access to the chamber and, where mechanical equipment is involved, to all functioning parts.

5.5.3.3. Ventilation

Where installed indoor, uncontaminated air shall be introduced continuously at a rate of 12 air changes per hour, or intermittently at a rate of 30 air changes per hour. Odour control facilities may also be warranted.

5.5.3.4. Electrical

Electrical equipment in grit removal areas where hazardous gases may accumulate shall comply with the Canadian Electrical Code and the regulations under the applicable provincial Power Standards.

5.5.3.5. Outside Facilities

Grit removal facilities located outside shall be protected from freezing.

5.5.3.6. Design Factors

5.5.3.6.1. Inlet

Inlet turbulence shall be minimized.

5.5.3.6.2. Type and Number of Units

Grit removal facilities (channel type) should have at least two hand-cleaned, or a mechanically cleaned unit with bypass. A single manually cleaned or mechanically cleaned grit chamber with bypass is acceptable for small sewage treatment plants serving separate sanitary sewer systems. Minimum facilities for larger plants serving separate sanitary sewers should be at least one mechanically cleaned unit with a bypass. Facilities other than channel-type are acceptable if provided with adequate and flexible controls for agitation and/or air supply devices and with grit collection and removal equipment.

5.5.3.6.3. Grit Channels

5.5.3.6.3.1. Velocity

Channel-type chambers shall be designed to provide controlled velocities as close as possible to 0.30 m/s for normal variation in flow.

5.5.3.6.3.2. Control Sections

Flow control sections shall be of the proportional or Sutro Weir type.

5.5.3.6.3.3. Channel Dimensions

The minimum channel width shall be 375 mm. The minimum channel length shall be that required to settle a 0.2 mm particle with a specific gravity of 2.65, plus a 50% allowance for inlet and outlet turbulence.

5.5.3.6.4. Grit Storage

With permanently positioned weirs, the weir crest should be kept 150 to 300 mm above the grit channel invert to provide for storage of settled grit (weir plates that are capable of vertical adjustment are preferred since they can be moved to prevent the sedimentation of organic solids following grit cleaning). Grit storage is also a function of the frequency of grit removal.

5.5.3.6.5. Detritus Tanks

Detritus tanks should be designed with sufficient surface area to remove a 0.2 mm, or smaller, particle with a specific gravity of 2.65 at the expected peak flow rate. Detritus tanks, since they are mechanically cleaned and do not need dewatering for cleaning, do not require multiple units, unless economically justifiable.

Separation of the organics from the grit before, during, or after the removal of the settled contents of the tank can be accomplished in one of the following ways:

1. The removed detritus can be washed in a grit washer with the organic laden wash water being returned to the head of the detritus tank;

2. A classifying-type conveyor can be used to remove the grit and return the organics to the detritus tank; or

3. The removed detritus can be passed through a centrifugal-type separator.

5.5.3.6.6. Aerated Grit Tanks

Aerated grit tanks for the removal of 0.2 mm, or larger, particles with specific gravity of 2.65, should be designed in accordance with the parameters in the following sub-sections.

5.5.3.6.6.1. Detention Time

Detention time shall be 2 to 5 minutes at the peak sewage flow rate.

5.5.3.6.6.2. Air Supply

Air supply rates should be in the range of 4.5 to 12.4 L/s per linear meter of tank. The higher rates should be used with tanks of large cross-section (i.e. greater than 3.6 m deep). Air supply should be via air diffusers (wide band diffusion header) positioned lengthwise along one wall of the tank, 600 to 900 mm above the tank bottom. Air supply should be variable.

5.5.3.6.6.3. Inlet Conditions

Inlet flow should be parallel to induced roll in tank. There shall be a smooth transition from inlet to circulation flow.

5.5.3.6.6.4. Baffling

A minimum of one transverse baffle near the outlet weir shall be provided. Additional transverse baffles in long tanks and longitudinal baffles in wide tanks should be considered.

5.5.3.6.6.5. Outlet Conditions

The outlet weir shall be oriented parallel to the direction of induced roll (i.e. at a right angle to the inlet).

5.5.3.6.6.6. Tank Dimensions

The lower limit of the above aeration rates are generally suitable for tanks up to 3.7 m deep and 4.3 m wide. Wider or deeper tanks require aeration rates in the upper end of the above range. Long, narrow aerated grit tanks are generally more efficient than short tanks and produce a cleaner grit. A length to width ratio of 2:5 to 5:1 is desirable. Depth to width ratios of 1:1.5 to 1:2 is acceptable.

5.5.3.6.6.7. Velocity

The surface velocity in the direction of roll in tanks should be 0.45 to 0.6 m/s (tank floor velocities will be approximately 75 per cent of above). The velocity across the floor of the tank shall not be less than 0.3 m/s.

5.5.3.6.6.8. Tank Geometry

"Dead spaces" in aerated grit tanks are to be avoided. Tank geometry is critical with respect to the location of the air diffusion header, sloping tank bottom, grit hopper and fitting of the grit collector mechanism into the tank structure. Consultation with Equipment Suppliers is advisable.

5.5.3.6.6.9. Multiple Units

Multiple units are generally not required unless economically justifiable, or where the grit removal method requires bypassing of the tank (as with clam shell bucket).

5.5.3.6.7. Mechanical Grit Chambers

Specific design parameters for mechanical grit chambers will be evaluated on a case-by-case basis.

5.5.3.6.8. Grit Washing

The need for grit washing should be determined by the method of final grit disposal.

5.5.3.6.9. Dewatering

Provision shall be made for isolating and dewatering each unit. The design shall provide for complete draining and cleaning by means of a sloped bottom equipped with a drain sump.

5.5.3.6.10. Water

An adequate supply of water under pressure shall be provided for cleanup.

5.5.3.7. Grit Removal

Grit facilities located in deep pits should be provided with mechanical equipment for pumping or hoisting grit to ground level. Such pits should have a stairway, approved-type elevator or manlift, adequate ventilation and adequate lighting.

5.5.3.8. Grit Handling

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

5.5.3.9. Grit Disposal

Disposal of grit shall be in accordance to the requirements of the DOEC.

5.5.4. Pre-aeration and Flocculation

Pre-aeration of raw wastewater, may be used to achieve one or more of the following objectives:

1. Odour control;
2. Grease separation and increased grit removal;
3. Prevention of septicity;
4. Grit separation;
5. Flocculation of solids;
6. Maintenance of DO in primary treatment tanks at low flows;
7. Increased removals of BOD and SS in primary units; and
8. Minimizes solids deposits on sidewalls and bottom of wet wells.

Flocculation of sewage with or without coagulating aids is worthy of consideration when it is desired to reduce the strength of sewage prior to subsequent treatment. Also, flocculation may be beneficial in pre-treating sewage containing certain industrial wastes.

5.5.4.1. Arrangement

The units should be designed so that removal from service will not interfere with normal operation of the remainder of the plant.

5.5.4.2. Pre-aeration

Figure 5.1 represents airflow requirements for different periods of pre-aeration. Pre-aeration periods should be 10 to 15 minutes if odour control and prevention of septicity are the prime objectives.

5.5.4.3. Flocculation

5.5.4.3.1. Detention Period

When air or mechanical agitation is used in conjunction with chemicals to coagulate or flocculate the sewage, the detention period should be about 30 minutes at the design flow. However, if polymers are used this may be varied.

5.5.4.3.2. Stirring Devices

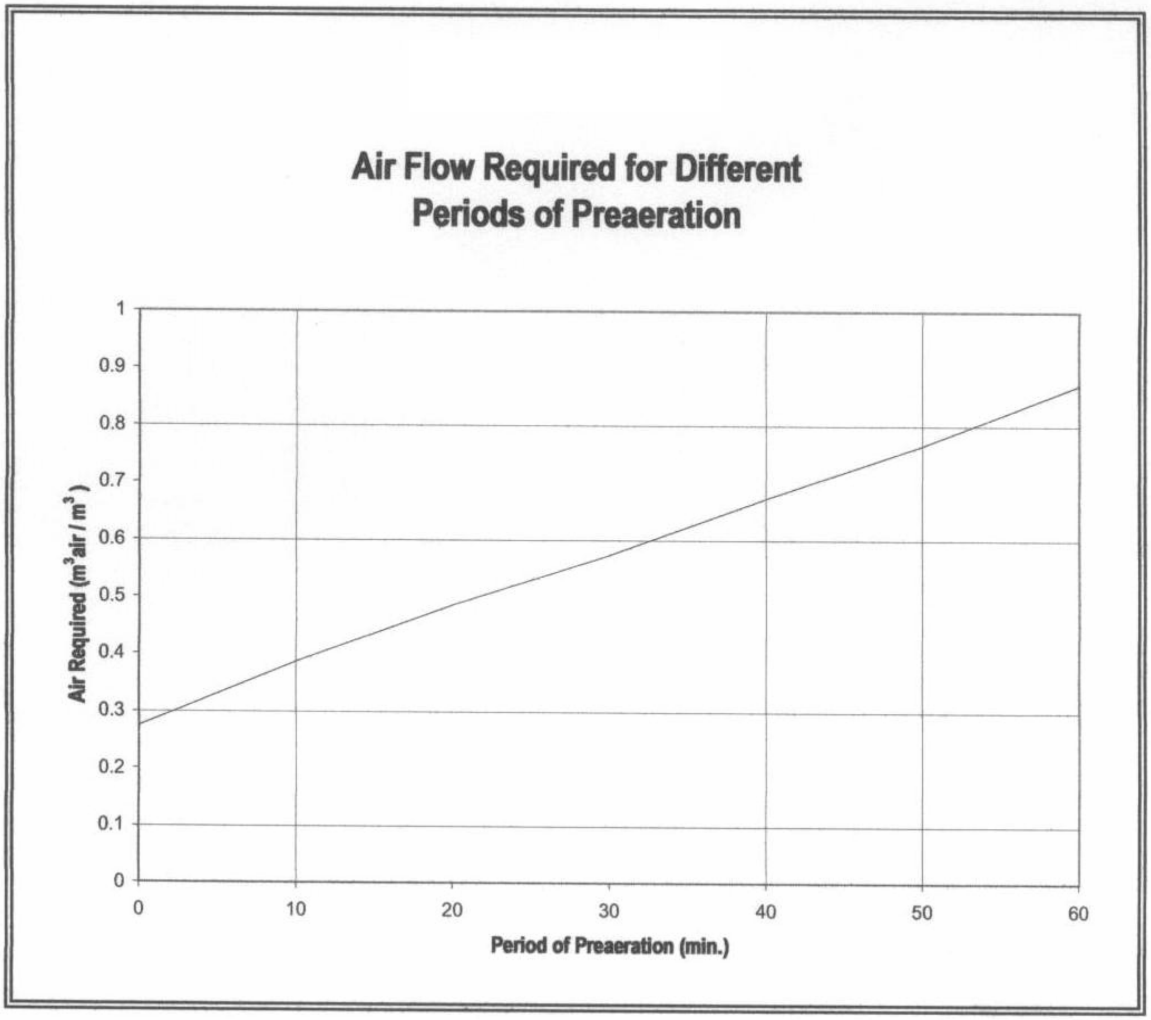
5.5.4.3.2.1. Paddles

Paddles should have a peripheral speed of 0.50 to 0.75 m/s to prevent deposition of solids.

5.5.4.3.2.2. Aerators

Any of the types of equipment used for aerating activated sludge may be utilized. It shall be possible to control agitation, to obtain good mixing and maintain self-cleaning velocities across the tank floor.

Figure 5.1



5.5.4.3.3. Details

Inlet and outlet devices should be designed to insure proper distribution and to prevent short-circuiting. Convenient means should be provided for removing grit.

5.5.4.3.4. Rapid Mix

At plants where there are two or more flocculation basins utilizing chemicals, provision shall be made for a rapid mix of the sewage with the chemical so that the sewage passing to the flocculation basins will be of uniform composition. The detention period provided in the rapid mixing chamber should be very short, 0.5 to 3.0 minutes.

5.5.5. Flow Equalization

Flow equalization can reduce the dry-weather variations in organic and hydraulic loadings at any wastewater treatment plant. It should be provided where large diurnal variations are expected.

5.5.5.1. Location

Equalization basins should be located downstream of pre-treatment facilities such as bar screens, comminutors and grit chambers.

5.5.5.2. Type

Flow equalization can be provided by using separate basins or on-line treatment units, such as aeration tanks. Equalization basins may be designed as either in-line or side-line units. Unused treatment units, such as sedimentation or aeration tanks, may be utilized as equalization basins during the early period of design life.

5.5.5.3. Size

Equalization basin capacity should be sufficient to effectively reduce expected flow and load variations to the extent deemed to be economically advantageous. With a diurnal flow pattern, the volume required to achieve the desired degree of equalization can be determined from a cumulative flow plot, or mass diagram, over a representative 24-hour period. To obtain the volume required to equalize the 24-hour flow:

1. Draw a line between the points representing the accumulated volume at the beginning and end of the 24-hour period. The slope of this line represents the average rate of flow.

2. Draw parallel lines to the first line through the points on the curve farthest from the first line; and

3. Draw a vertical line between the lines drawn in No. 2. The length of this line represents the minimum required volume.

5.5.5.4. Operation

5.5.5.4.1. Mixing

Where applicable, aeration or mechanical equipment shall be provided to maintain adequate mixing. Corner fillets and hopper bottoms with draw-offs should be provided to alleviate the accumulation of sludge and grit.

5.5.5.4.2. Aeration

Where applicable, aeration equipment shall be sufficient to maintain a minimum of 1.0 mg/L of dissolved oxygen in the mixed basin contents at all times. Air supply rates should be a minimum of 0.15 L/s per cubic meter storage capacity. The air supply should be isolated from other treatment plant aeration requirements to facilitate process aeration control, although process air supply equipment may be utilized as a source of standby aeration.

5.5.5.4.3. Controls
Inlets and outlets for all basin compartments shall be suitably equipped with accessible external valves, stop plates, weirs or other devices to permit flow control and the removal of an individual unit from service. Facilities shall also be provided to measure and indicate liquid levels and flow rates.

5.5.5.5. Electrical
All electrical work in housed equalization basins shall comply with the Canadian Electrical Code and the regulations under applicable Provincial Power Standards.

5.5.5.6. Access
Suitable access shall be provided to facilitate cleaning and the maintenance of equipment.

5.6. Clarification

5.6.1. Sedimentation Tanks

5.6.1.1. Design Requirements
The need for and the design of primary sedimentation tanks will be influenced by various factors, including the following:

1. The characteristics of the raw wastewater; the type of sludge digestion systems, either available or proposed (aerobic digestion should not be used with raw primary sludges);

2. The presence, or absence, of secondary treatment following primary treatment;

3. The need for handling of waste activated sludge in the primary settling tank; and

4. The need for, or possible economic benefits through, phosphorus removal in the primary settling tank(s).

5.6.1.1.1. Number of Units
Multiple units capable of independent operation are desirable and shall be provided in all plants where design flows exceed 500 m^3/d. Plants not having multiple units shall include other provisions to assure continuity of treatment.

5.6.1.1.2. Arrangement of Units
Settling tanks shall be arranged in accordance with Section 5.4.3.7.

5.6.1.1.3. Interaction with Other Processes
1. Pumping directly to any clarifier is prohibited, unless special provision is included in the design of pump controls. Attention should be focused so that pumps deliver smooth flow transmissions at all times, with a minimal energy gradient.

2. For activated sludge plants employing high-energy aeration, provisions should be made for floc to be reformed before settling.

3. For primary clarifiers, tanks and equipment must be sized to not only accommodate raw waste solids but also those solids introduced by thickener overflows, anaerobic digester overflow and sometimes waste activated sludge.

5.6.1.1.4. Flow Distribution and Control

Effective flow measurement devices and control appurtenances (i,e., valves, gates, splitter boxes, etc.) shall be provided to permit proper proportion of flow to each unit. Parallel basins should be of the same size, otherwise flow shall be distributed in proportion to surface area.

5.6.1.1.5. Tank Configuration and Proportions

Consideration should be given to the probable flow pattern in the selection of tank size and shape, and inlet and outlet type and location. Generally rectangular clarifiers are designed with length-to-width ratios of at least 4:1, and width to depth ratios of 1:1 and 2.25:1.

5.6.1.1.6. Site Constraints

The selection of feasible clarifier alternatives should include the following site considerations:
1. Wind direction;
2. Proximity to residents;
3. Soil conditions
4. Groundwater conditions; and
5. Available space.

5.6.1.1.7. Size Limitations

Rectangular clarifiers shall have a maximum length of 90 m. Circular clarifiers shall have a maximum diameter of 60 m. The minimum length of flow from inlet to outlet shall be 3 m, unless special provisions are made to prevent short-circuiting. The vertical sidewater depth shall be designed to provide an adequate separation zone between the sludge blanket and the overflow weirs.

5.6.1.1.8. Inlet Structures

Inlet structures should be designed to dissipate the inlet velocity, to distribute the flow equally and to prevent short-circuiting. Channels should be designed to maintain a velocity of at least 0.3 m/s at one-half design average flow. Corner pockets and dead ends should be eliminated and corner fillets or channelling used where necessary. Provisions shall be made for elimination or removal of floating materials in inlet structures.

5.6.1.1.9. Outlet Arrangements

Overflow weirs shall be adjustable for levelling, and sufficiently long to avoid high heads, which result in updraft currents.

5.6.1.1.9.1. Location

Overflow weirs shall be located to optimize actual hydraulic detention time and minimize short-circuiting. Peripheral weirs shall be placed at least 0.3 m from the wall.

5.6.1.1.9.2. Weir Troughs
Weir troughs shall be designed to prevent submergence at maximum design flow and to maintain a velocity of at least 0.3 m/s at one-half design average flow.

5.6.1.1.10. Submerged Surfaces
The tops of troughs, beams and similar submerged construction elements shall have a minimum slope of 1.4 vertical to 1 horizontal; the underside of such elements should have a slope of 1:1 to prevent the accumulation of scum and solids.

5.6.1.1.11. Unit Dewatering
Unit dewatering features shall conform to the provisions outlined in Section 5.4.4.2 (Unit Dewatering, Flotation Protection and Plugging). The bypass design should also provide for redistribution of the plant flow to the remaining units.

5.6.1.1.12. Freeboard
Walls of settling tanks shall extend at least 150 mm above the surrounding ground surface and shall provide not less than 300 mm freeboard. Additional freeboard or the use of wind screens is recommended where larger settling tanks are subject to high velocity wind currents that would cause tank surface waves and inhibit effective scum removal or excessive heat loss resulting in freezing problems.

5.6.1.1.13. Clarifier Covers
Clarifiers may be required to be covered for winter operation. The structure should be constructed with adequate headroom for easy access. The structure must include adequate lighting, ventilation and heating. Humidity and condensation shall be controlled inside the structure.

5.6.1.1.14. Surface Settling Rates
5.6.1.1.14.1. Primary Settling Tanks
Units should be designed for a detention period of 1.0 hour at peak flow and a surface settling rate of approximately 61 $m^3/(m^2.d)$ at peak flows, assuming activated sludge wasting followed by aeration. Other rates and detention periods shall be considered to meet special conditions such as dictated by the secondary treatment process, character of waste, or to adjust the efficiency to meet specific requirements. Primary settling of normal domestic sewage can be expected to remove 30 to 35% of the influent BOD. However, anticipated BOD removal for sewage containing appreciable quantities of industrial wastes (or chemical additions to be used) should be determined by laboratory tests and consideration of the quantity and character of the wastes.

5.6.1.1.14.2. Intermediate Settling Tanks
Surface settling rates for intermediate settling tanks following series units of fixed film reactor processes should not exceed 61 $m^3/(m^2.d)$ at peak flow.

5.6.1.1.14.3. Final Settling Tanks

Settling tests should be conducted where a pilot study of biological treatment is warranted by unusual waste characteristics or treatment requirements. Testing shall be done where proposed loadings exceed the limits sets forth in this section:

1. **Fixed Film Biological Reactors**

   Surface settling rates for settling tanks following trickling filters or rotating biological contactors should not exceed 49 $m^3/(m^2.d)$ at peak flow.

2. **Activated Sludge**

   The hydraulic loadings should not exceed 49 $m^3/(m^2.d)$ at peak flow with no phosphorus removal and 40 $m^3/(m^2.d)$ at peak flow with phosphorus removal. The solids loading for all activated sludge processes should not exceed 244 $kg/(m^2.d)$ at the peak flows. Consideration should be given to flow equalization

5.6.1.2. Types Of Settling

5.6.1.2.1. Type I Settling (Discrete Settling)

Type I settling is assumed to occur in gravity grit chambers handling wastewater and in basins used for preliminary settling (silt removal) of surface waters. A determination of the settling velocity of the smallest particle to be 100% removed is fundamental to the design of Type I clarifiers. Because each particle is assumed to settle independently and with a constant velocity, a mathematical development is possible, based on Newton's Law and Stoke's Law.

5.6.1.2.2. Type II Settling (Flocculant Settling)

Type II settling occurs when particles initially settle independently but flocculate as they proceed the depth of the tank. As a result of flocculation, the settling velocities of the aggregates formed change with time, and a strict mathematical solution is not possible. Laboratory testing is required to determine appropriate values for design parameters. Type II settling can occur during clarification following fixed-film processes, primary clarification of wastewater, and clarification of potable water treated with coagulants.

Type II settling can also occur above the sludge blanket in clarifiers following activated sludge treatment; however design procedures based on Type III settling are normally used to design these units.

5.6.1.2.3. Type III Settling (Hindered or Zone Settling)

Type III settling occurs in clarifiers following activated sludge processes and gravity thickeners. While Type II processes may occur to a limited extent in such units, it is Type III that governs design. In suspensions undergoing hindered settling, the solids concentration is usually much higher than in discrete or flocculant processes. As a result, the contacting particles tend to settle as a zone or blanket, and maintain the same position relative to each other.

5.6.1.2.4. Settling Type Design Criteria

Table 5.6 outlines design parameters for sedimentation tanks based upon their associated settling type.

Table 5.6
Sedimentation Basins Design Parameters

Type of Settling	Sidewater Depth (m)	Surface Overflow (1) Rate ($m^3/m^2.d$)	Weir Loading Rate ($m^3/m.d$)(1)(2)	Solids Loading (kg/m^2d)
Type I and II	3.0 – 4.6	≤ 40 at design average flow ≤ 60 at peak hourly flow	125 – 370	N/A
Type III	3.5 – 4.6	**1) Settling following activated sludge:** ≤ 30 at design average flow ≤ 50 at peak hourly flow **2) Settling following extended aeration:** ≤ 15 at design average flow ≤ 35 at peak hourly flow **3) Settling following fixed film processes:** ≤ 25 at design average flow ≤ 45 at peak hourly flow **4) Settling following separate nitrification:** ≤ 35 at peak hourly flow	< 250(4)	49 (at SVI of 300) -290 (at SVI of 100)

(1)When several different overflow criteria are given (design average flow, peak hourly flow) the clarifier area to be used in the design is the larger of those computed in each case.
(2)At design average flow.
(3)If pumping is required, weir loading rate should be related to pump delivery rates to avoid short-circuiting.
(4)Where weirs are located so that density currents upturn below them, the rate should not exceed 186 $m^3/m.d$.

5.6.1.3. Scum and Sludge Removal

5.6.1.3.1. Scum Removal

Effective scum collection and removal facilities, including baffling, shall be provided for all settling tanks. Scum baffles are to be placed ahead of the outlet weirs and extend 300 mm below the water surface. The unusual characteristics of scum, which may adversely affect pumping, piping, sludge handling and disposal, should be recognized in design. Provisions may be made for the discharge of scum with the sludge; however, other special provisions for disposal may be necessary.

5.6.1.3.2. Sludge Removal

Sludge collection and withdrawal facilities shall be designed to assure rapid removal of the sludge and minimization of density currents. Suction withdrawal should be provided for activated sludge plants designed for reduction of the nitrogenous oxygen demand and is encouraged for those plants designed for carbonaceous oxygen demand reduction. Each settling tank shall have its own sludge withdrawal lines to insure adequate control of the sludge wasting rate for each tank.

5.6.1.3.2.1. Sludge Collection
Sludge collection mechanisms shall remain in operation during sludge withdrawal. Mechanism speeds shall be such as to avoid undue agitation while still producing desired collection results.

5.6.1.3.2.2. Sludge Hopper
The minimum slope of the sidewalls shall be 1.7 vertical to 1 horizontal. Hopper wall surfaces should be made smooth with rounded corners to aid in sludge removal. Hopper bottoms shall have a maximum dimension of 0.6 m. Extra depth sludge hoppers for sludge thickening are not acceptable. The hoppers are to be accessible for sounding and cleaning.

5.6.1.3.2.3. Cross-Collectors
Cross-collectors, serving one or more settling tanks, may be useful in place of multiple sludge hoppers.

5.6.1.3.2.4. Sludge Removal Piping
Each hopper shall have an individually valved sludge withdrawal line at least 150 mm in diameter. The static head available for withdrawal of sludge shall be 750 mm or greater, as necessary to maintain a 1.0 m/s velocity in the withdrawal line. Clearance between the end of the withdrawal line and the hopper walls shall be sufficient to prevent 'bridging" of the sludge. Adequate provisions shall be made for rodding or back-flushing individual pipe runs. Piping shall also be provided to return waste sludge to primary clarifiers.

5.6.1.3.2.5. Sludge Removal Control
Sludge wells equipped with telescoping valves or other appropriate equipment shall be provided for viewing, sampling and controlling the rate of sludge withdrawal from each tank hopper. The use of easily maintained sight glass and sampling valves may be appropriate. A means of measuring the sludge removal rate from each hopper shall be provided. Air lift type of sludge removal will not be approved for removal of primary sludge. Sludge pump motor control systems shall include time clocks and valve activators for regulating the duration and sequencing of sludge removal.

5.6.2. Enhanced Primary Clarification

5.6.2.1. Chemical Enhancement
Chemical coagulation of raw wastewater before sedimentation promotes flocculation of finely divided solids into more readily settleable flocs, thereby increasing SS, BOD, and phosphorus removal efficiencies. Sedimentation with coagulation may remove 60 to 90% of the total suspended solids (TSS), 40 to 70% of the BOD, 30 to 60% of the chemical oxygen demand (COD), 70 to 90% of the phosphorus, and 80 to 90% of the bacteria loadings. In comparison, sedimentation without coagulation may remove only 40 to 70% of the TSS, 25 to 40% of the BOD, 5 to 10% of the phosphorus loadings, and 50 to 60% of the bacteria loading. Additional information on the selection and application of chemicals for phosphorus removal is included in Section 5.14 dealing with tertiary treatment and nutrient removal.

Advantages of coagulation include greater removal efficiencies, the ability to use higher overflow rates, and more consistent performance. Disadvantages of coagulation include an increased mass of primary sludge, production of solids that are often more difficult to thicken and dewater, and an increase in operational cost and operator attention. The designer of chemical coagulation facilities should consider the effect of enhanced primary sedimentation on downstream solids-processing facilities.

5.6.2.1.1. Chemical Coagulants.

Historically, iron salts, aluminum salts, and lime have been the chemical coagulants used for wastewater treatment. Iron salts have typically been the most common of the coagulants used for primary treatment. Only a few plants use lime as a coagulant for primary treatment since lime addition produces more primary sludge because of the chemical solids than do metals salts and lime is more difficult to store, handle, and feed. Coagulant selection for enhanced sedimentation should be based on performance, reliability, and cost. Performance evaluation should use jar tests of the actual wastewater to determine dosages and effectiveness. Operating experience, cost, and other relevant information drawn from other plants should be considered during selection. Organic polymers are sometimes used as flocculation aids.

5.6.2.1.2. Rapid Mix

During rapid mix, the first step of the coagulation process, chemical coagulants are mixed with the raw wastewater. The coagulants destabilize the colloidal particles by reducing the forces (zeta potential), keeping the particles apart, which allows their agglomeration. The destabilization process occurs within seconds of coagulant addition. At the point of chemical addition, intense mixing will ensure uniform dispersion of the coagulant throughout the raw wastewater. The intensity and duration of mixing must be controlled, however, to avoid over-mixing or under-mixing. Over-mixing may reduce the removal efficiency by breaking up existing wastewater solids and newly formed floc. Under-mixing inadequately disperses the chemical, increases chemical use, and reduces the removal efficiency.

The velocity gradient, G, is a measure of mixing intensity. Velocity gradients of 300 s^{-1} are typically sufficient for rapid mix, but some designers have recommended velocity gradients as high as 1000 s^{-1}. Mechanical mixers, in-line blenders, pumps, baffled compartments, baffled pipes, or air mixers can accomplish rapid mix. The mixing intensity of mechanical mixers and in-line blenders is independent of flow rate, but these mixers cost considerably more than other types and might become clogged or entangled with debris. Air mixing eliminates the problem of debris and can offer advantages for primary sedimentation, especially if aerated channels or grit chambers already exist.

Pumps, Parshall flumes, flow distribution structures, baffled compartments, or baffled pipes, methods often used for upgrading existing facilities, offer a lower-cost but less-efficient alternative to separate mixers for new construction. Methods listed above are less efficient than separate mixers because, unlike separate mixing, the mix intensity depends on the flow rate.

5.6.2.1.3. Flocculation

During the flocculation step of the coagulation process, destabilized particles grow and agglomerate to form large, settleable flocs. Through gentle prolonged mixing, chemical bridging and/or physical enmeshment of particles occur. Flocculation is slower and more dependent on time and agitation than is the rapid-mix step. Typical detention times for flocculation range between 20 and 30 minutes. Aerated and mechanical grit chambers, flow distribution structures, and influent wells are areas that promote flocculation upstream of primary sedimentation. Advantages and disadvantages of different configurations resemble those for rapid-mix facilities.

Like rapid mix, the velocity gradient, G, achieved with each configuration should be checked. Velocity gradients should be maintained from 50 to 80 s^{-1}. Polymers are sometimes added during the flocculation step to promote floc formation. Polymers should enter as dilute solution to ensure thorough dispersion of polymers throughout the wastewater. Polymers may provide a good floc with only turbulence and detention in the sedimentation tank inlet distribution.

5.6.2.1.4. Coagulant Addition

Supplementing conventional primary sedimentation with chemical coagulation requires minimal additional construction. The optimal point for coagulant addition is as far upstream as possible from primary sedimentation tanks. The optimum feed point for coagulant addition often varies from plant to plant. If possible, several different feed points should be considered for additional flexibility. Dispersing the coagulant throughout the wastewater is essential to minimize coagulant dosage and concrete and metal corrosion associated with coagulant addition. Flow-metering devices should be installed on chemical feed lines for dosage control.

5.6.2.2. Plate and Tube Settlers

Plate and tube settlers are utilized to increase the effective settling area within the clarifier or settling basin. They can be used with or without chemical enhancement but typically are utilized in advanced primary applications. These types of settlers operate on the principle that by increasing the area where particles can settle within the settling unit through the use of inclined tubes or plates will result in reduced footprint units accomplishing equivalent overflow rates to conventional settling basins with a much greater water surface area.

5.6.2.2.1. Calculation of Settling Area

The settling area within a plate clarifier is equal to the horizontally projected area of the vertically inclined plates. Therefore a settling basin equipped with (n) plates of overall surface area (A) inclined at an angle (θ) from the horizontal will have an equivalent settling area which can be calculated utilizing the equation:

$$Total\,Settling\;Area = n\,A\left(\cos\theta\right)$$

Overflow rates can then be calculated utilizing the total settling area rather than the water surface area of the unit. Similar principles can be utilized for the calculation of total surface area and surface overflow rates for tube settlers.

5.6.2.2.2. Configuration

Typical settling plates are approximately 0.2 - 0.6 m wide and 3 m long with 50 mm spacing between multiple plates. Plate settlers are designed to operate in the laminar flow regime. Plate spacing must be large enough to prevent scouring of settled solids by the upward flowing liquid, to transport solids in a downward direction to the sludge hoppers, and to avoid plugging between the plates. In some instances plate vibrators or mechanical scrapers can be utilized to prevent plugging. Flash mixers and flocculation chambers may be required ahead of the plate clarifier (as with all clarifiers) to mix in chemicals to promote floc growth and enhance the clarification process. Care must be taken to transport flocculated feed to the settling unit at less than 0.3 m/s to prevent floc break-up.

5.6.2.3. Ballasted Floc Clarifiers

The ballasted flocculation and settling process is a precipitative process, which utilizes micro-sand combined with polymer for improved floc attachment and thus improved settling. The process involves:

1. Coagulation;
2. Injection;
3. Maturation; and
4. Sedimentation.

During the coagulation process, metal-salt coagulants (typically alum or ferric sulphate) are added and thoroughly mixed into solution. The water then enters the injection chamber where polymer addition is followed by micro-sand injection and subsequent flash mixing. The maturation process acts like a typical flocculation chamber, utilizing an optimum mixing energy for optimized floc agglomeration onto the micro-sand.

In the settling process, water enters the lower region of the basin and travels through lamella plates. Solids collection with tube settlers in the bottom of the settling chamber is followed by cyclonic separation of micro-sand and sludge.

The micro-sand exiting the hydrocyclone is then re-injected into the treatment process. The micro-sand used typically has a diameter of 50 to 100 microns. The typical detention times for coagulation, injection, and maturation are 1 to 2 minutes, 1 to 2 minutes, and 4 to 6 minutes, respectively. The detention time of the settling basin depends on the rise rate, which is typically between 50 to 100 m/d.

5.6.3. Dissolved Air Flotation

Dissolved air flotation (DAF) refers to the process of solids-liquids separation caused by the introduction of fine gas (usually air) bubbles to the liquid phase. The bubbles attach to the solids, and the resultant buoyancy of the combined solids-gas matrix causes the matrix to rise to the surface of the liquid where it is collected by a skimming mechanism.

Flotation can be employed in both liquid clarification and solids concentration applications. Flotator liquid effluent (known as subnatant) quality is the primary performance factor in clarification applications. These applications include flotation of refinery, meatpacking, meat rendering, and other “oily” wastewaters. Float-solids concentrations are the main performance criteria in solids concentration flotation applications. Concentration applications include the flotation of waste solids of biological, mining, and metallurgical processes.

5.6.3.1. Process Design Considerations and Criteria

The feed solids to a DAF clarifier are typically mixed with a pressurized recycle flow before tank entry. The recycle flow is typically DAF tank effluent, although providing water from another source, as a backup is often advisable if poor DAF performance causes an effluent high in SS. The recycle flow is pumped to an air saturation tank where compressed air enters and dissolves into the recycle. As the pressurized recycle containing dissolved air is admitted back into the DAF tank (its surface is at atmospheric pressure), the pressure release from the recycle forms the air bubbles for flotation. A typical bubble-size distribution contains bubbles diameters ranging from 10 to 100 µm. Solids and air particles float and form a blanket on the DAF tank surface while the clarified effluent flows under the tank baffle and over the effluent weir. In general, the blanket on top of the DAF tank will be 150 to 300 mm thick.

Chemical conditioning with polymers is frequently used to enhance DAF performance. Polymer use significantly increases applicable solids-loading rates and solids capture but less effectively increases float-solids concentrations. If a polymer is used, it generally is introduced at the point where the recycle flow and the solids feed are mixed. Introducing the polymer solution into the recycle just as the bubbles are being formed and mixed with the solids produces the best results. Good mixing to ensure chemical dispersion while minimizing shearing forces will provide the best solids-air bubble aggregates.

Numerous factors affect DAF process performance, including:

1. Type and characteristics of feed solids;
2. Hydraulic loading rate;
3. Solids-loading rate;
4. Air-to-solids ratio;
5. Chemical conditioning;
6. Operating policy;
7. Float-solids concentration; and
8. Effluent clarity.

5.6.3.1.1. Types of Solids

A variety of solids can be effectively removed by flotation. Among these are conventional activated sludge, solids from extended aeration and aerobic digestion, pure-oxygen activated sludge, and dual biological (trickling filter plus activated-sludge) processes.

Effects of the DAF process factors listed in the previous section make it difficult to document the specific performance characteristics of each of these types of solids. In other words, the specific conditions at each plant (for example, types of process, SRT, and SVI in the aeration basin) dictate DAF performance to a greater extent than can be compensated for by flotation equipment adjustments such as air-to-solids ratio.

5.6.3.1.2. Hydraulic Loading Rate

Hydraulic loading rate refers to the sum of the feed and recycle flow rates divided by the net available flotation area. Dissolved air flotation clarifiers typically are designed for hydraulic loading rates of 60 to 120 m/d, assuming no use of conditioning chemicals. The additional turbulence in flotators when the hourly hydraulic loading rate exceeds 5 m/h may hinder the establishment of a stable float blanket and reduce the attainable float-solids turbulence forces the flow regime away from plug flow and more toward mixed flow. The addition of a polymer flotation aid generally is required to maintain satisfactory performance at hourly hydraulic loading rates greater than 5 m/h.

5.6.3.1.3. Solids-Loading Rate

The solids-loading rate of a DAF clarifier is generally denoted in terms of weight of solids per effective flotation area. With the addition of polymer, the solids-loading rate to a DAF thickener generally can be increased 50 to 100%, with up to a 0.5 to 1% increase in the thickened-solids concentration.

Operational difficulties may arise when the solids-loading rate exceeds approximately 10 kg/m^2h. The difficulties generally are caused by coincidental operation of excessive hydraulic loading rates and by float-removal difficulties. Even in those instances when the hydraulic-loading rate can be maintained at less than 120 m/d, operation at solids-loading rates more than 10 kg/m^2h can cause float-removal difficulties. The increased amount of float created at high solids-loading rates necessitates continuous skimming, often at high skimming speeds.

Increased skimming speed, however, can cause float blanket disturbance and increase the amount of solids in the subnatant to unacceptable levels. In these circumstances, the addition of polymer flotation aid to increase the rise rate of the solids and the rate of float-blanket consolidation can alleviate some of the operating difficulties. Although stressed conditions, such as mechanical breakdown, excessive solids wastage, or adverse solids characteristics, may make it necessary to periodically operate in this manner, the flotation system should not be designed on this basis.

5.6.3.1.4. Feed-Solids Concentration

Changes in feed-solids concentration indirectly affect flotation in connection with the resultant changes in operating conditions. If the fed flow rate, recycle flow, pressure, and skimmer operations remain constant, an increase in feed-solids concentration results in a decrease in the air-to-solids ratio. Changes in feed-solids concentration also result in changes to the float-blanket inventory and depth. Adjustments to the float skimmer speed many be required when operating strategy includes maintenance of a specific float-blanket depth or range of depths.

5.6.3.1.5. Air-to-Solids Ratio

The air-to-solids ratio is perhaps the single most important factor affecting DAF performance. It refers to the weight ratio of air available for flotation to the solids to be floated in the feed stream. Reported ratios range from 0.01:1 to 0.4:1; adequate flotation is achieved in most municipal wastewater clarification applications at ratios of 0.02:1 to 0.06:1. Pressurization system sizing depends on many variables, including design solids loading, pressurization system efficiency, system pressure, liquid temperature, and concentration of dissolved solids. Pressurization system efficiencies differ among manufacturers and system configurations and can range from as low as 50% to more than 90%. Detailed information is available regarding the design, specification, and testing of pressurization systems.

Because the float from a DAF clarifier contains a considerable amount of entrained air, this pumping application requires positive-displacement or centrifugal pumps that do not air bind, and special consideration of suction conditions. Initial density of the skimmed solids is approximately 700 kg/m^3. After the solids are held for a few hours, the air escapes and the solids return to normal densities. Float-solids content increases with increasing air-to-solids ratios up to a point where further increases in air-to-solids ratios result in only a nominal or no increase in float solids. The typical air-to-solids ratio at which float solids are maximized varies from 2 to 4%.

5.6.3.1.6. Float-Blanket Depth

The float produced during the flotation process must be removed from the flotation tank. The float-removal system usually consists of a variable-speed float skimmer and a beach arrangement. The volume of float that must be removed during each skimmer pass depends on the solids-loading rate, the chemical dosage rate, and the consistency of the float material.

Float-removal system skimmers are designed and operated to maximize float drainage time by incrementally removing only the top (driest) portion of the float and preventing the float blanket from expanding to the point where float exits the system in the subnatant. The optimal float depth varies from installation to installation. A float depth of 0.8 to 1.5 cm is typically sufficient to maximize float-solids content.

5.6.3.1.7. Polymer Addition

Chemical conditioning can enhance the performance of a DAF unit. Conditioning agents can be used to improve clarification and/or increase the float-solids concentration attainable with the unit. The amount of conditioning agent required, the point of addition (in the feed stream or recycle stream), and the method for intermixing should be specifically determined for each installation. Bench-scale flotation tests or pilot-unit tests provide the most effective method of determining the optimal chemical conditioning scheme for a particular installation. Typical polymer doses range from 2 to 5 g dry polymer/kg dry feed solids.

The addition of polymer usually affects solids capture to a greater extent than float-solids content. The float-solids content generally is increased up to 0.5% by the addition of dry polymer at a dosage of 2 to 5 g/kg dry solids.

If the lower ranges of hydraulic and solids loadings are used, the addition of polymer flotation aid typically is unnecessary for well-designed and operated DAF clarifiers. Maintenance of proper design and operating conditions as described in the preceding sections results in stable operation and satisfactory performance in terms of solids capture and float-solids concentration.

Solids recovery without polymer addition generally will be much greater than 90% when the DAF unit is sized as previously discussed. High loadings or adverse solids conditions can reduce solids recovery to 75 to 90%. Polymer-aided recovery can exceed 95%.

Under normal operations, the solids recycled from the DAF unit will not be damaging to the treatment system but will have the effect of increasing the WAS to be processed. In cases where the solids or hydraulic loading already are excessive, the recycled solids pose an additional burden on the system. Polymers should be employed under these conditions to maximize solids capture from the DAF unit.

5.6.4. Protective and Service Facilities

5.6.4.1. Operator Protection

All clarification tanks shall be equipped to enhance safety for operators. Such features shall appropriately include machinery covers, lifelines, stairways, walkways, handrails and slip-resistant surfaces.

5.6.4.2. Mechanical Maintenance Access

The design shall provide for convenient and safe access to routine maintenance items such as gearboxes, scum removal mechanisms, baffles, weirs, inlet stilling baffle area, and effluent channels.

5.6.4.3. Electrical Fixtures and Controls

Electrical fixtures and controls in enclosed settling basins shall comply with the Canadian Electrical Code and the applicable Provincial Power Standards. The fixtures and controls shall be located so as to provide convenient and safe access for operation and maintenance. Adequate area lighting shall be provided.

5.7. Sludge Handling and Disposal

Sludge handling and disposal must be considered as an integral part of any complete sewage treatment system. The following is a summary of the sludge handling and disposal options and the various process and treatment requirements best suited to the option selected. Re-use and recovery alternatives of sludge by-products are also included as disposal options.

Plans and specifications for sludge handling disposal must be incorporated in the design of all sewage treatment facilities.

5.7.1. Process Selection

The selection of sludge handling unit processes should be based upon at least the following considerations:

1. Local land use;
2. System energy requirements;
3. Cost effectiveness of sludge thickening and dewatering;
4. Equipment complexity and staffing requirements;
5. Adverse effects of heavy metals and other sludge components upon the unit processes;
6. Sludge digestion or stabilization requirements;
7. Side streams or return flow treatment requirements (e.g. digester or sludge storage facilities supernatant, dewatering unit filtrate, wet oxidation return flows);
8. Sludge storage requirements;
9. Methods of ultimate disposal; and
10. Back-up techniques of sludge handling and disposal.

5.7.2. Sludge Conditioning

5.7.2.1. Chemical Conditioning

5.7.2.1.1 Chemical Requirements

Chemical conditioning methods involve the use of organic or inorganic flocculants to promote the formation of a porous, free draining cake structure. The ranges of some chemical conditioning requirements are outlined in Table 5.7.

Table 5.7
Chemical Conditioning Requirements

Sludge	$FeCl_3$ (kg/tonne dry solids)	$Ca(OH)_2$ (kg/tonne dry solids)	Polymers (kg/tonne dry solids)
RP	10 – 30	0 – 50	1.5 – 2.5
R(P + TF)	30 – 60	0 – 150	2 – 5
R(P + AS)	40 – 80	0 – 150	3 – 7.5
AS	60 – 100	50 – 1500	4 – 12.5
DP	20 – 30	30 – 80	1.5 – 4
D(P + TF)	40 – 80	50 – 150	3 – 7.5
D(P + AS)	60 - 100	50 -150	3 - 10

R = Raw P = Primary TF = Trickling Filter AS = Activated Sludge D = Digested

5.7.2.1.2. Laboratory Testing

The selection of the most suitable chemical(s) and the actual dosage requirements for sludge conditioning shall be determined by full-scale testing.

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

5.7.2.1.3. Conditioning Chemicals

With most thickening operations and with belt filter press dewatering operations the most commonly used chemicals are polymers. For dewatering by vacuum filtration, ferric salts, often in conjunction with lime, are most commonly used, although with centrifuge dewatering, chemical conditioning using polymers is most prevalent, with metal salts being avoided mainly due to corrosion problems. The ultimate disposal methods may also have an effect on the choice of conditioning chemicals. For instance, lime and ferric compounds should be avoided with incineration options.

5.7.2.1.3.1. Iron or Aluminum Salts

Most raw sludge can be filtered with ferric salts alone, although digested sludge will require an addition of lime with the ferric salt. The lime:ferric chloride ratio is typically 3:1 to 4:1 for best results. If metallic salts are used without lime, the resulting low pH sludge will be highly corrosive to carbon steel and shall require materials such as plastic, stainless steel, or rubber for proper handling.

5.7.2.1.3.2. Lime

Hydrated limes, both the high calcium and dolomitic types, can be used for sludge conditioning in conjunction with metal salts or alone.

5.7.2.1.3.3. Polymers

Polymers used for sludge conditioning are long-chain water-soluble organic molecules of high molecular weight. They are used in wastewater suspensions to cause flocculation through adsorption. Equipment for polymer addition must be able to withstand potential corrosion.

5.7.2.1.3.4. Chemical Feed System

The chemical feed system shall be paced at the rate of sludge flow to the dewatering unit. The chemical feed system should be either close to the dewatering unit or controllable from a point near the dewatering unit. Sufficient mixing shall be provided so as to disperse the conditioner throughout the sludge. The chemical feed rates should allow for at least a 10:1 range of chemical flow to the dewatering unit.

5.7.2.2. Heat Conditioning

Heat conditioning of sludge consists of subjecting the sludge to high levels of heat and pressure. Heat conditioning can be accomplished by either a non-oxidative or oxidative system. Heat conditioning high temperatures cause hydrolysis of the encapsulated water-solids matrix and lysing of the biological cells. The hydrolysis of the water matrix destroys the gelatinous components of the organic solids and thereby improves the water-solids separation characteristics.

5.7.2.2.1. Operating Temperatures and Pressures

Typical operating temperatures range from150 to 260 °C. Operating pressures range from 1100 to 2800 kPa. Typical sludge detention times vary between 15 and 60 minutes.

5.7.2.2.2. Increase in Aeration Tank Organic Loading

Although the heat conditioning system has been proven to be an effective sludge conditioning technique for subsequent dewatering operations, the process results in a significant organic loading to the aeration tanks of the sewage treatment plant if supernatant is returned to the aeration system. This is due to the solubilization of organic matter during the sludge hydrolysis. This liquor can represent 25 to 50% of the total loading on the aeration tanks and allowances must be made in the treatment plant design to accommodate this loading increase.

5.7.2.2.3. Design Considerations

5.7.2.2.3.1. Materials

Heat conditioning results in the production of extremely corrosive liquids requiring the use of corrosion-resistant materials for the liquid handling.

5.7.2.2.3.2. Sludge Grinding

Sludge grinders shall be provided to macerate the sludge to a particle size less than 6 mm to prevent fouling of the heat exchangers.

5.7.2.2.3.3. Feed Pumps

Feed pumps shall be capable of discharging sludge at pressures of 1400 to 2800 kPa and must be resistant to abrasion.

5.7.2.2.3.4. Heat Exchangers

The efficiency of the heat exchangers is dependent on the transfer coefficients and the temperature differences of the incoming and outgoing sludge.

5.7.2.2.3.5. Reaction Vessel

The reaction vessel shall be of sufficient volume to provide for a sludge detention time of 15 to 60 minutes. The detention time depends on the sludge characteristics, temperature and the level of hydrolysis required.

5.7.2.2.3.6. Hot Water Re-circulation Pump

The hot water re-circulation pump shall be capable of handling hot water at a temperature of 25 to 60 ˚C.

5.7.2.2.3.7. Odour Control
Heat conditioning, particularly the non-oxidative process, can result in the production of odorous gases in the decant tank. If ultimate sludge disposal is via incineration, these gases can be incinerated in the upper portion of the furnace. If incineration is not a part of the sludge handling process, a catalytic or other type of oxidating unit should be used.

5.7.2.2.3.8. Solvent Cleaning
Scale formation in the heat exchangers, pipes and reaction vessel require acid washing equipment to be provided.

5.7.2.2.3.9. Piping
All the high pressure piping for the sludge heat conditioning system shall be tested at a pressure of 3500 kPa. Low-pressure piping shall be tested at 1.5 times the working pressure, or 1400 kPa, whichever is greater.

5.7.2.2.3.10. Decant Tank
The decant tank functions as a storage and sludge consolidation unit. The tank should be covered and provided with venting and a deodorization arrangement. The tank should be designed using loadings of 245 kg/m^2d for primary sludge and 145 kg/m^2d for biological sludges. The underflow will range from 10 to 15 % TS.

5.7.2.2.4. Laboratory Testing
Since process efficiency is dependent on achieving a degree of solubilization (hydrolysis) that reduces the specific resistant to an acceptable range, batch testing with a laboratory autoclave should be employed. This procedure permits accurate control of the time and temperature functions affecting the level of hydrolysis. The level of solubilization is determined from the loss of TSS during heat treatment.

5.7.2.3. Addition of Admixtures
Another common form of physical conditioning is the addition of admixtures such as fly ash, incinerator ash, diatomaceous earth, or waste paper. These conditioning techniques are most commonly used with filter presses or vacuum filters. The admixtures when added in sufficient quantities produce a porous lattice structure in the sludge which results in decreased compressibility and improved filtering characteristics. When considering such conditioning techniques, the beneficial and detrimental effects of the admixture on such parameters as overall sludge mass, calorific value, etc., must be evaluated along with the effects on improved solids content.

5.7.3. Sludge Thickening

5.7.3.1. Applicability

As the first step of sludge handling, the need for sludge thickeners to reduce the volume of sludge should be considered.

The design of thickeners (gravity, dissolved-air flotation, centrifuge and others) should consider the type and concentration of sludge, the sludge stabilization processes, the method of ultimate sludge disposal, chemical needs and the cost of operation. Particular attention should be given to the pumping and piping of the concentrated sludge and possible onset of anaerobic conditions. Sludge thickening to at least 5% solids prior to transmission to digesters should be considered. Wherever possible, pilot-plant and/or bench-scale data should be used for the design of sludge thickening facilities. With new plants, this may not always be possible and, in such cases, empirical design parameters must be used. The following subsections outline the normal ranges for the design parameters of such equipment.

In considering the need for sludge thickening facilities, the designer should evaluate the economics of the overall treatment processes, with and without facilities for sludge water content reduction. This evaluation should consider both capital and operating costs of the various plant components and sludge disposal operations affected.

5.7.3.2. Multiple Units

With sludge thickening equipment, multiple units will generally be required unless satisfactory sludge storage facilities or alternate sludge disposal methods are available for use during periods of equipment repair. Often the need for full standby units will be unnecessary if the remaining duty units can be operated for additional shifts in the event of equipment breakdown.

5.7.3.3. Thickener Location

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

1. Prior to digestion for raw primary, excess activated sludge or mixed sludge;
2. Prior to dewatering facilities;
3. Following digestion for sludge or supernatant; or
4. Following dewatering facilities for concentration of filtrate, decant, centrate, etc.

Where thickeners are to be housed, adequate ventilation shall be provided.

5.7.3.4. Thickening Methods and Performance with Various Sludge Types

The commonly employed methods of sludge thickening and their suitability for the various types of sludge are shown in Table 5.8. In selecting a design figure for the thickened sludge concentration, the designer should keep in mind that all thickening devices are adversely affected by high Sludge Volume Indices (SVI's) and benefited by low SVI's in the influent activated sludge. The ranges of thickened sludge concentrations given in Table 5.8 assume an SVI of approximately 100.

Table 5.8
Sludge Thickening Methods and Performance with Various Sludge Types

Thickening Method	Sludge Type	Performance Expected
Gravity	Raw Primary	Good, 8–10% Solids
	Raw Primary and Waste Activated	Poor, 5-8% Solids
	Waste Activated	Very Poor, 2-3% Solids (better results reported for oxygen excess activated sludge)
	Digested Primary	Very Good, 8-14% Solids
	Digested Primary and Waste Activated	Poor, 6-9% Solids
Dissolved Air Flotation	Waste Activated (not generally used for other sludge types)	Good, 4-6% Solids and ≥95% Solids Capture with Flotation Aids
Centrifugation	Waste Activated	8-10% and 80-90% Solids Capture with Basket Centrifuges; 4-6% and 80-90% Solids Capture with Disc-nozzle Centrifuges; 5-8% and 70-90% Solids Capture with Solid Bowl Centrifuges

5.7.3.5. Sludge Pretreatment

Wherever thickening devices are being installed, special consideration must be given to the need for sludge pretreatment in the form of sludge grinding to avoid plugging pumps, lines and thickening equipment. Sludge conditioning by chemical conditioning is also considered as a type of pretreatment.

5.7.3.6. Gravity Thickening

5.7.3.6.1 Process Application

Gravity thickening is principally used for primary sludge, and mixtures of primary and waste activated sludges, with little use for waste activated sludges alone. Due to the better performance of other methods for waste activated sludges, gravity thickening has limited application for such sludges.

5.7.3.6.2. Design Criteria

5.7.3.6.2.1 Tank Shape

The gravity thickener shall be circular in shape.

5.7.3.6.2.2. Tank Dimensions

Typical maximum tank diameters should range between 21 and 24 m. Sidewater depth shall be between 3 and 3.7 m.

5.7.3.6.2.3. Floor Slope

The acceptable range for gravity sludge thickener floor slopes is 2:12 to 3:12.

5.7.3.6.2.4. Solids Loading

The type of sludge shall govern the design value for solids loading to the gravity thickener. Table 5.9 outlines recommended solids loading values.

Table 5.9
Solids Loading on Gravity Thickeners for Various Sludge Types

Type of Sludge	Solids Load (kg/m^2d) Acceptable Range
Primary	95 – 120
Waste Activated	12 – 40
Modified Activated	50 –100
Trickling Filter	40 –50

Solids loading for any combination of primary sludge and waste activated sludge shall be based on a weighted average of the above loading rates. Use of metal salts for phosphorus removal may affect the solids loading rates.

5.7.3.6.2.5. Dilution

Improved thickening is achieved by diluting sludge to 0.5 to 1% solids because that dilution reduces the interface between the settling particles. Primary sewage effluent or secondary effluent may be utilized to dilute sludge before thickening.

5.7.3.6.2.6. Hydraulic Overflow Rate

The hydraulic overflow rate shall be kept sufficiently high to prevent septic conditions from developing in the thickener. The acceptable ranges for overflow rates are as follows:

Primary Sludge 0.28-0.38 L/m^2s
Secondary Sludge 0.22-0.34 L/m^2s
Mixture 0.25-0.36 L/m^2s

5.7.3.6.2.7. Sludge Volume Ratio

The sludge volume ratio (SVR) is defined as the volume of the sludge blanket divided by the daily volume of sludge (underflow) pumped from the thickener. Though deeper sludge blankets and longer SVR are desirable for maximum concentrations, septic conditions due to anaerobic biodegradation on warmer months limit the upper values of SVR to about 2 days.

Recommended SVR values, in days, are as follows:
1. Warmer months – 0.3 to 1; and
2. Colder months – 0.5 to 2.

5.7.3.6.2.8. Hydraulic Retention Time
A minimum of 6-hour detention of liquid is required. For maximum compaction of the sludge blanket, 24 hours is the recommended time required. During peak conditions, the retention time may have to be shortened to keep the sludge blanket depth below the overflow weirs, thus, preventing excessive solids carry-over.

5.7.3.6.2.9. Sludge Underflow Piping
The length of suction lines should be kept as short as possible. Consideration should be given to the use of dual sludge withdrawal lines.

5.7.3.6.2.10. Chemical Conditioning
Provision should be made for the addition of conditioning chemicals into the sludge influent lines (polymers, ferric chloride or lime are the most likely chemicals to be used to improve solids capture).

5.7.3.6.2.11. Mechanical Rake
The mechanical rake should have a tip speed of 50 to 100 mm/s. The rake shall be equipped with hinged-lift mechanisms when handling heavy sludges such as lime treated primary sludge. The use of a surface skimmer is recommended.

5.7.3.6.2.12. Overflow Handling
The normal quality of thickener overflow (also known as thickener overhead or supernatant) is about the same as raw sewage quality. Consequently, returning the overflow to primary settling tank or aeration tank should not present any operational problem.

Direct recycling of thickener overflow to the grit chamber, primary settling tank, trickling filter, RBC or aeration tank is permitted. The supernatant shall not be discharged into the secondary settling tank, disinfection tank, sewer outfall, or receiving water.

5.7.3.7. Air Flotation
5.7.3.7.1. Applicability
Unlike heavy sludges, such as primary and mixtures of primary and excess activated sludges, which are generally most effectively thickened in gravity thickeners, light excess activated sludges can be successfully thickened by flotation. In general, air flotation thickening can be employed whenever particles tend to float rather, than sink. These procedures are also applied if the materials have a long subsidence period and resist compaction for thickening by gravity.

The advantages of air flotation compared with gravity thickeners for excess activated sludges include its reliability, production of higher sludge concentrations, and better solids capture. Its disadvantages include the need for greater operating skill and higher operating costs.

5.7.3.7.2. Pilot Scale Testing
Experience has shown that flotation operations cannot be designed on the basis of purely mathematical formulations or by the use of generalized design parameters, and therefore some bench-scale and/or pilot-scale testing will be necessary.

5.7.3.7.3. Design Parameters

The following design parameters are given only as a guide to indicate the normal range of values experienced in full-scale operations.

5.7.3.7.3.1. Recycle Ratio

The recycle ratio varies with suppliers and typically falls between 0 and 500% of the influent flow. Recycled flows may be pressurized up to 520 kPa.

5.7.3.7.3.2. Air to Solids Weight Ratio

Typical air to solids weight ratios shall be between 0.02 and 0.05.

5.7.3.7.3.3. Feed Concentration

Feed concentration of activated sludge (including recycle) to the flotation compartment should not exceed 5000 mg/L.

5.7.3.7.3.4. Hydraulic Feed Rate

Where the hydraulic feed rate includes influent plus recycle, the flotation units shall be designed hydraulically to operate in the range of 0.3 to 1.5 L/m^2s. A maximum hydraulic loading rate of 0.5 L/m^2s shall be adhered to when no coagulant aids are used to improve flotation. The feed rate should be continuous rather than on/off.

5.7.3.7.3.5. Solids Loading

Without any addition of flocculating chemicals, the solids loading rate for activated sludge to a flotation unit should be between 40 and 100 kg/m^2d. With the proper addition of flocculating chemicals, the solids loading rate may be increased to 240 kg/m^2d. These loading rates will generally produce a thickened sludge of 3 to 5 % total solids.

5.7.3.7.3.6. Chemical Conditioning

Chemicals used, as coagulant aids shall be fed directly to the mixing zone of the feed sludge and recycle flow.

5.7.3.7.3.7. Detention Time

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

5.7.3.7.4. Thickened Sludge Withdrawal

The surface skimmer shall move thickened sludge over the dewatering beach into the sludge hopper. Either positive displacement, or centrifugal pumps, which will not air bind, should be used to transfer sludge from the hopper to the next phase of the process. In selecting pumps, the maximum possible sludge concentrations should be taken into consideration.

5.7.3.7.5. Bottom Sludge

A bottom collector to move draw off settled sludge into a hopper must be provided. Draw off from the hopper may be by gravity or pumps.

5.7.3.8. Centrifugation

5.7.3.8.1. Types of Centrifuges

Three types of centrifuges may be utilized for sludge thickening. These include the solid bowl conveyor, disc-nozzle and basket centrifuges.

5.7.3.8.2. Applicability

To date, there has only been limited application of centrifuges for sludge thickening, despite their common use for sludge dewatering. As thickening devices, their use has been generally restricted to excess activated sludges. In the way of general comments, the following are given:

1. Centrifugal thickening operations can have substantial maintenance and operating costs;
2. Where space limitations, or sludge characteristics make other methods unsuitable, or where high-capacity mobile units are needed, centrifuges have been used; and
3. Thickening capacity, thickened sludge concentration and solids capture of a centrifuge are greatly dependent on the SVI of the sludge.

5.7.3.8.3. Solids Recovery

The most suitable operating range is generally 85 – 95% solids recovery.

5.7.3.8.4. Polymer Feed Range

A polymer feed range of 0 to 4.0 g/kg of dry solids is generally acceptable.

5.7.4. Anaerobic Sludge Digestion

5.7.4.1. Applicability

Anaerobic digestion may be considered beneficial for sludge stabilization when the sludge volatile solids content is 50% or higher and if no inhibitory substances are present or expected. Anaerobic digestion of primary sludge is preferred over activated sludge because of the poor solids-liquid separation characteristics of activated sludges. Combining primary and secondary sludges will result in settling characteristics better than activated sludge but less desirable than primary alone. Chemical sludges containing lime, alum, iron, and other substances can be successfully digested if the volatile solids content remains high enough to support the biochemical reactions and no toxic compounds are present. If an examination of past sludge characteristics indicates wide variations in sludge quality, anaerobic digestion may not be feasible because of its inherent sensitivity to changing substrate quality. The following is a list of sludges, which are suitable for anaerobic digestion:

1. Primary and lime;
2. Primary and ferric chloride;
3. Primary and alum;
4. Primary and trickling filter;
5. Primary, trickling filter, and alum;
6. Primary and waste activated;
7. Primary, waste activated, and lime;
8. Primary, waste activated, and alum;
9. Primary, waste activated, and ferric chloride; and
10. Primary, waste activated, and sodium aluminate.

5.7.4.1.1. Advantages

The advantages offered by anaerobic digestion include:

1. Excess energy over that required by the process is produced. Methane is produced and can be used to heat and mix the reactor. Excess methane gas can be used to heat space or produce electricity, or as engine fuel;

2. The quantity of total solids for ultimate disposal is reduced. The volatile solids present are converted to methane, carbon dioxide, and water thereby reducing the quantity of solids. About 30 to 40% of the total solids may be destroyed and 40 to 60% of the volatile solids may be destroyed;

3. The product is a stabilized sludge that may be free from strong or foul odours and can be used for land application as ultimate disposal because the digested sludge contains plant nutrients;

4. Pathogens are destroyed to a high degree during the process. Thermophilic digestion enhances the degree of pathogen destruction; and

5. Most organic substances found in municipal sludge are readily digestible except lignins, tannins, rubber, and plastics.

5.7.4.1.2. Disadvantages

The disadvantages associated with anaerobic digestion include:

1. The digester is easily upset by unusual conditions and erratic or high loadings and very slow to recover;

2. Operators must follow proper operating procedures;

3. Heating and mixing equipment are required for satisfactory performance;

4. Large reactors are required because of the slow growth of methanogens and required solid retention times (SRT's) of 15 to 20 days for a high-rate system. Thus capital cost is high.

5. The resultant supernatant side stream is a strong waste stream that greatly adds to the loading of the wastewater plant. It contains high concentrations of BOD, COD, suspended solids and ammonia nitrogen;

6. Cleaning operations are difficult because of the closed vessel. Internal heating and mixing equipment can become major problems as a result of corrosion and wear in harsh inaccessible environments.

7. A sludge poor in dewatering characteristics is produced;

8. The possibility of explosion as a result of inadequate operation and maintenance, leaks, or operator carelessness exists; and

9. Gas line condensation or clogging can cause major maintenance problems.

5.7.4.2. Digestion Tanks and Number of Stages

With anaerobic sludge digestion facilities, the need for multiple units can often be avoided by providing two-stage digestion along with sufficient flexibility in sludge pumpage and mixing so that one stage can be serviced while the other stage receives the raw sludge pumpage. Single stage digesters will generally not be satisfactory due to the usual need for sludge storage, and effective supernating. They will be considered, however, where the designer can show that the above concerns can be satisfied and that alternate means of sludge processing or emergency storage can be used in the event of breakdown.

5.7.4.3. Access Manholes

At least two, 1-meter diameter access manholes should be provided in the top of the tank in addition to the gas dome. There should be stairways to reach the access manholes. A separate sidewall manhole shall be provided. The opening should be large enough to permit the use of mechanical equipment to remove grit and sand. This manhole shall be located near the bottom of the sidewall. All manholes shall be provided with gas-tight and watertight covers.

5.7.4.4. Safety

Non-sparking tools, safety lights, rubber-soled shoes, safety harness, gas detectors for inflammable and toxic gases and at least two self-contained breathing units shall be provided for workers involved in cleaning the digesters.

Necessary safety facilities shall be included where sludge gas is produced. All tank covers shall be provided with pressure and vacuum relief valves and flame traps together with automatic safety shut-off valves. Water seal equipment shall not be installed.

5.7.4.5. Field Data

Wherever possible, such as in the case of plant expansions, actual sludge quantity data should be considered for digester design. Often, due to errors introduced by poor sampling techniques, inaccurate flow measurements or unmeasured sludge flow streams, the sludge data from existing plants may be unsuitable for use in design. Therefore, before sludge data is used for design, it should be assessed for its accuracy.

5.7.4.6. Typical Sludge Qualities and Generation Rates for Different Unit Processes

When reliable data are not available, the sludge generation rates and characteristics given in Table 5.10 may be used.

Table 5.10
Typical Sludge Qualities and Generation Rates

Unit Process	Liquid Sludge	Solids Concentration		Volatile Solids	Dry Solids	
	(L/m^3)	Range (%)	Average (%)	(%)	(g/m^3)	(g/cap.d)
Primary Sedimentation with Anaerobic Digestion						
Undigested (no P removal)	2.0	1.5 – 8	5.0	65	120	55
Undigested (with P removal)	3.2	3.5 – 7	4.5	65	170	77
Digested (no P removal)	1.1	5 – 13	6.0	50	75	34
Digested (with P removal)	1.6	5 - 13	5.0	50	110	50
Primary Sedimentation and Conventional Activated Sludge with Anaerobic Digestion						
Undigested (no P removal)	4.0	2 – 7	4.5	65	160	62
Undigested (with P removal)	5.0	2 – 6.5	4.0	60	220	100
Digested (no P removal)	2.0	2 – 6	5.0	50	115	52
Digested (with P removal)	3.5	2 – 6	4.0	45	150	68
Contact Stabilization and High Rate with Anaerobic Digestion						
Undigested (no P removal)	15.5	0.4–2.6	1.1	70	170	77
Undigested (with P removal)	19.1	0.4-2.6	1.1	60	210	95
Digested (no P removal)	6.1	1 – 3	1.9	70	115	52
Digested (with P removal)	8.1	1 - 3	1.9	60	155	70
Extended Aeration with Aerated Sludge Holding Tank						
Waste Activated (no P removal)	10.0	0.4-1.9	0.9	70	90	41
Waste Activated (P removal)	13.3	1.4-1.9	0.9	60	120	55
Sludge Holding Tank (no P removal)	4.0	1.4-5.0	2.0	70	80	36
Sludge Holding Tank (P removal)	5.5	0.4-4.5	2.0	60	110	50

Note:
1. L/m^3 denotes litres of liquid sludge per cubic metre of treated sewage
2. g/m^3 denotes grams of dry solids per cubic metre of treated sewage
3. the above values are based on typical raw sewage with total BOD = 570 mg/L, soluble BOD = 50, SS = 200 mg/L, P = 7 mg/L, NH_3 = 20 mg/L

5.7.4.7. Solids Retention Time

The minimum solids retention time for a low rate digester shall be 30 days. The minimum solids retention time of a high rate digester shall be 15 days.

5.7.4.8. Design of Tank Elements

5.7.4.8.1. Digester Shape

Anaerobic digesters are generally cylindrical in shape with inverted conical bottoms. Heat loss from digesters can be minimized by choosing a proper depth-diameter ratio, such that the total surface area is the least for a given volume. A cylinder with diameter equal to depth can be shown to be the most economical shape from heat loss viewpoint. However, structural requirements and scum control aspects also govern the optimum depth-diameter ratio.

5.7.4.8.2. Floor Slope

To facilitate draining, cleaning and maintenance, the following features are desirable:

1. The tank bottom should slope to drain toward the withdrawal pipe;

2. For tanks equipped with mechanisms for withdrawal of sludge, a bottom slope not less than 1:12 (vertical:horizontal) is recommended; and

3. Where the sludge is to be removed by gravity alone, 1:4 slope is recommended.

5.7.4.8.3. Depth and Freeboard

For those units proposed to serve as supernatant development tanks, the depth should be sufficient to allow for the formation of a reasonable depth of supernatant liquor. A minimum water depth of 6 m is recommended. The acceptable range for sidewater depth is between 6 and 14 m.

The freeboard provided must take into consideration the type of cover and maximum gas pressure. For floating covers, the normal working water level in the tank under gas pressure is approximately 0.8 m below the top of the wall, thus providing from 0.5 to 0.6 m of freeboard between the liquid level and the top of the tank wall. For fixed flat slab roofs, a freeboard of 0.3 to 0.6 m above the working liquid level is commonly provided. For fixed conical or domed roofs, the freeboard between the working liquid level and the top of the wall inside the tank can be reduced to less than 0.3 m.

5.7.4.8.4. Scum Control

Scum accumulation can be controlled by including any of the following provisions in the equipment design:

1. Floating covers keep the scum layer submerged and thus moist and more likely to be broken up;

2. Discharging re-circulated sludge on the scum mat serves the same purpose as (1);

3. Re-circulating sludge gas under pressure through the tank liquors and scum;

4. Mechanically destroying the scum by employing rotating arms or a propeller in a draft tube;

5. A large depth-area ratio; or

6. A concentrated sludge feed to the digester.

Items (5) and (6) would release large volumes of gas per unit area, keep the scum in motion and mix the solids in the digester.

5.7.4.8.5. Grit and Sand Control

The digesters should be designed to minimize sedimentation of the particles and facilitate removal if settling takes place. These objectives can be achieved if tank contents are kept moving at 0.23 to 0.3 m/s and the floor slopes are about 1:4.

5.7.4.8.6. Alkalinity and pH Control

The effective pH range for methane producers is approximately 6.5 to 7.5 with an optimum range of 6.8 to 7.2. Maintenance of this optimum range is important to ensure good gas production and to eliminate digester upsets.

The stability of the digestion process depends on the buffering capacity of the digester contents; the ability of the digester contents to resist pH changes. The alkalinity is a measure of the buffer capacity of a freshwater system. Higher alkalinity values indicate a greater capacity for resisting pH changes. The alkalinity shall be measured as bicarbonate alkalinity. Values for alkalinity in anaerobic digesters range from 1500 to 5000 mg/L as $CaCO_3$. The volatile acids produced by the acid producers tend to depress pH. Volatile acid concentrations under stable conditions range from 100 to 500 mg/L. Therefore, a constant ratio below 0.25 of volatile acids to alkalinity shall be maintained so that the buffering capacity of the system can be maintained.

Sodium bicarbonate, lime, sodium carbonate, and ammonium hydroxide application are recommended for increasing alkalinity of digester contents.

5.7.4.8.7. Mixing

Thorough mixing via digester gas (compressor power requirement 5 to 8 W/m^3) or mechanical means (6.6 W/m^3) in the primary stage will be necessary in all cases when digesters are proposed. This mixing shall assure the homogeneity of the digester contents, and prevent stratification.

Gas mixing methods are preferred. Gas mixing may be accomplished in any one of the following manners:

1. Short mixing tubes;
2. One or more deep-draft tubes;
3. Diffusers at the digester floor; or
4. Gas discharge below scum level.

5.7.4.8.8. Sludge Inlets, Outlets, Re-circulation, and High Level Overflow

5.7.4.8.8.1. Multiple Inlets and Draw-Offs

Multiple sludge inlets and draw-offs and, where used, multiple re-circulation suction and discharge points to facilitate flexible operation and effective mixing of the digester contents, shall be provided unless adequate mixing facilities are provided within the digester.

5.7.4.8.8.2. Inlet Configurations

One inlet should discharge above the liquid level and be located at approximately the centre of the tank to assist in scum break-up. The second inlet should be opposite to the suction line at approximately the 0.7 diameter point across the digester.

5.7.4.8.8.3. Inlet Discharge Location

Raw sludge inlet discharge points should be so located as to minimize short-circuiting to the digester sludge or supernatant draw-offs.

5.7.4.8.8.4. Sludge Withdrawal
Sludge withdrawal to disposal should be from the bottom of the tank. The bottom withdrawal pipe should be interconnected with the necessary valving to the re-circulation piping, to increase operational flexibility in mixing the tank contents.

5.7.4.8.8.5. Emergency Overflow
An unvalved vented overflow shall be provided to prevent damage to the digestion tank and cover in case of accidental overfilling. This emergency overflow shall be piped to an appropriate point and at an appropriate rate in the treatment process or sidestream treatment facilities to minimize the impact on process units.

5.7.4.8.9. Primary Tank Capacity
The primary digestion tank capacity should be determined by rational calculations based upon such factors as volume of sludge added, its percent solids and character, the temperature to be maintained in the digesters, the degree or extent of mixing to be obtained and the degree of volatile solids reduction required. Calculations shall be submitted to justify the basis of design.

When such calculations are not based on the above factors, the minimum primary digestion tank capacity outlined in Sections 5.7.4.8.9.1 and 5.7.4.8.9.2 will be required. Such requirements assume that a raw sludge is derived from ordinary domestic wastewater, that a digestion temperature is to be maintained in the range of 32 °C to 39 °C, that 40 to 50% volatile matter will be maintained in the digested sludge and that the digested sludge will be removed frequently from the system.

5.7.4.8.9.1. High Rate Digester
The primary high rate digester shall provide for intimate and effective mixing to prevent stratification and to assure homogeneity of digester content. The system may be loaded at a rate up to 1.6 kg of volatile solids per cubic meter of volume per day in the active digestion unit. When grit removal facilities are not provided, the reduction of digester volume due to grit accumulation should be considered.

5.7.4.8.9.2. Low Rate Digester
For low rate digesters where mixing is accomplished only by circulating sludge through an external heat exchanger, the system may be loaded up to 0.64 kg of volatile solids per cubic meter of volume per day in the active digestion unit. This loading may be modified upward or downward depending upon the degree of mixing provided.

5.7.4.8.10. Secondary Digester Sizing
The secondary digester should be sized to permit solids settling for decanting and solids thickening operations, and in conjunction with possible off-site facilities, to provide the necessary digested sludge storage. The necessary total storage time will depend upon the means of ultimate sludge disposal, with the greatest time required with soil conditioning operations (winter storage), and with less storage required with landfilling or incineration ultimate disposal methods. Offsite storage in sludge lagoons, sludge storage tanks, or other facilities may be used to supplement the storage capacity of the secondary digester. If high-rate primary digesters are

used and efficient dewatering within the secondary digester is required, the secondary digester must be conservatively sized to allow adequate solids separation (secondary to primary sizing ratios of 2:1 to 4:1 are recommended).

5.7.4.8.11. Digester Covers

To provide gas storage volume and to maintain uniform gas pressures, a separate gas storage sphere should be provided, or at least one digester cover should be of the gas-holder floating type. If only one floating cover is provided, it shall be on the secondary digester. Insulated pressure and vacuum relief valves and flame traps shall be provided. Access manholes and at least two 200 mm sampling wells should also be provided on the digester covers. Steel is the most commonly used material for digester covers. However, other properly designed and constructed materials can also be successfully employed, such as concrete and fibreglass.

5.7.4.8.12. Sludge Piping

Maximum flexibility should be provided in terms of sludge transfer from primary and secondary treatment units to the digesters, between the primary and secondary digesters, and from the digesters to subsequent sludge handling operations. The minimum diameter of sludge pipes shall be 200 mm for gravity withdrawal and 150 mm for pump suction and discharge lines. Provision should be made for flushing and cleaning sludge piping. Sampling points should be provided on all sludge lines. Main sludge transfer lines should be from the bottom of the primary digester to the mid-point of the secondary digester. Additional transfer lines should be from intermediate points in the primary digester (these can be dual-purpose supernatant and sludge lines).

5.7.4.8.13. Overflows

Each digester should be equipped with an emergency overflow system.

5.7.4.9. Gas Collection, Piping and Appurtenances

All portions of the gas system including the space above the tank liquor, storage facilities and piping shall be so designed that under all normal operating conditions, including sludge withdrawal, the gas will be maintained under positive pressure. All enclosed areas where any gas leakage might occur shall be adequately ventilated. All gas collection equipment, piping and appurtenances shall comply with the Canadian Gas Association Standard B105-M93.

5.7.4.9.1. Safety Equipment

All necessary safety facilities shall be included where gas is produced. Pressure and vacuum relief valves and flame traps together with automatic safety shut-off valves, are essential. Water seal equipment shall not be installed. Gas safety equipment and gas compressors should be housed in a separate room with an exterior entrance.

Provision should also be made for automatically purging the combustion chamber of the heating unit thoroughly with air after a shutdown or pilot light failure, and before it can be ignited. This will provide certainty that no explosive mixture exists within the unit.

5.7.4.9.2. Gas Piping and Condensate

The main gas collector line from the digestion tanks shall be at least 64 mm in diameter with the gas intake being well above the digester scum level, generally at least 1.2 m above the maximum liquid level in the tank. If gas mixing is used, the gas withdrawal pipe must be of sufficient size to limit the pressure drop in terms of the total gas flow from the digester. Such flow includes not only the daily gas production, but also the daily gas recycling flow. The recycling gas flow information should be combined with the estimate peak daily gas flow data to determine the proper piping size.

Gas pipe slopes of 20 mm/m are desirable with a minimum slope of 10 mm/m for drainage. The maximum velocity in sludge-gas piping shall be limited to not more than 3.7 m/s.

Gas piping shall slope to condensation traps at low points. The use of float controlled condensate traps is not permitted.

Adequate pipe support is essential to prevent breaking, and special care should be given where pipes are located underground.

Gas piping and pressure relief valves must include adequate flame traps. They should be installed as close as possible to the device serving as a source of ignition.

5.7.4.9.3. Gas Utilization Equipment

Gas burning boilers, engines, etc., should be located at ground level and in well ventilated rooms, not connected to the digester gallery. Gas lines to these units shall be provided with suitable flame traps.

5.7.4.9.4. Electrical Systems

Electrical fixtures and controls, in places enclosing anaerobic digestion appurtenances, where hazardous gases are normally contained in the tanks and piping, shall comply with the Canadian Electrical Code, Part 1 and the applicable provincial power standards. Digester galleries should be isolated from normal operating areas.

5.7.4.9.5. Waste Gas

Waste gas burners shall be readily accessible and should be located at least 15 m away from any plant structure if placed at ground level, or may be located on the roof of the control building if sufficiently removed from the tank. In remote locations it may be permissible to discharge the gas to the atmosphere through a return-bend screened vent terminating at least 3 m above the walking surface, provided the assembly incorporates a flame trap. Waste gas burners shall be of sufficient height and so located to prevent injury to personnel due to wind or downdraft conditions.

All waste gas burners shall be equipped with automatic ignition, such as a pilot light or a device using a photoelectric cell sensor. Consideration should be given to the use of natural or propane gas to insure reliability of the pilot light.

Provision for condensate removal, pressure control, and flame protection ahead of waste burners is always required.

5.7.4.9.6. Ventilation

Any underground enclosures connecting with digestion tanks or containing sludge or gas piping or equipment shall be provided with forced ventilation in accordance with Section 5.3.10. Tightly fitting self-closing doors should be provided at connecting passageways and tunnels to minimize the spread of gas.

5.7.4.9.7. Meter

A gas meter with bypass shall be provided, to meter total gas production for each active digestion unit. A single gas meter with proper interconnected gas piping may measure total gas production for two-stage digestion systems operated in series. Where multiple primary digestion units are utilized with a single secondary digestion unit, a gas meter shall be provided for each primary digestion unit. The secondary digestion unit may be interconnected with the gas measurement unit of one of the primary units. Interconnected gas piping shall be properly valved with gas tight gate valves to allow measurement of gas production from either digestion unit or maintenance of either digestion unit.

Gas meters may be of the orifice plate, turbine or vortex type. Positive displacement meters should not be utilized. The meter must be specifically designed for contact with corrosive and dirty gases.

5.7.4.10. Digestion Tank Heating

5.7.4.10.1. Heating Capacity

Sufficient heating capacity shall be provided to consistently maintain the design sludge temperature considering insulation provisions and ambient cold weather conditions. Where digestion tank gas is used for other purposes, an auxiliary fuel may be required.

5.7.4.10.2. Insulation

Wherever possible, digestion tanks should be constructed above ground-water level and should be suitably insulated to minimize heat loss.

5.7.4.10.3. Heating Facilities

Sludge may be heated by circulating the sludge through external heaters or by heating units located inside the digestion tank. The external heat exchanger systems are preferred.

5.7.4.10.3.1. External Heating

Piping shall be designed to provide for the preheating of feed sludge before introduction to the digesters. Provisions shall be made in the layout of the piping and valving to facilitate heat exchanger tube removal and cleaning of these lines. Heat exchanger sludge piping shall be sized for peak heat transfer requirements. Heat exchangers should have a heating capacity of 130 % of the calculated peak-heating requirement to account for the occurrence of sludge tube fouling.

5.7.4.10.3.2. Other Heating Methods

1. The use of hot water heating coils affixed to the walls of the digester, or other types of internal heating equipment that require emptying the digester contents for repair, are not acceptable.

2. Other systems and devices have been developed recently to provide both mixing and heating of anaerobic digester contents. These systems will be reviewed on their own merits. Operating data detailing their reliability, operation, and maintenance characteristics will be required.

5.7.4.10.4. Hot Water Internal Heating Controls

5.7.4.10.4.1. Mixing Valves

A suitable automatic mixing valve shall be provided to temper the boiler water with return water so that the inlet water to the removable heat jacket or coils in the digester can be held below a temperature at which caking will be accentuated. Manual control should also be provided by suitable by-pass valves.

5.7.4.10.4.2. Boiler Controls

The boiler should be provided with suitable automatic controls to maintain the boiler temperature at approximately 82 ºC to minimize corrosion and to shut off the main gas supply in the event of pilot burner or electrical failure, low boiler water level, excessive temperature, or low gas pressure.

5.7.4.10.4.3. Boiler Water Pumps

Boiler water pumps shall be sealed and sized to meet the operating conditions of temperature, operating head, and flow rate. Duplicate units shall be provided.

5.7.4.10.4.4. Thermometers

Thermometers should be provided to show inlet and outlet temperatures of sludge, hot water feed, hot water return and boiler water.

5.7.4.10.4.5. Water Supply

The chemical quality should be checked for suitability for this use.

5.7.4.10.5. External Heater Operating Controls

All controls necessary to insure effective and safe operation are required. Provision for duplicate units in critical elements should be considered.

5.7.4.11. Supernatant Withdrawal

5.7.4.11.1. Piping Size

Supernatant piping should not be less than 150 mm in diameter. Precaution must be taken to avoid loss of digester gas through supernatant piping.

5.7.4.11.2. Withdrawal Arrangement

5.7.4.11.2.1. Withdrawal Levels

Piping should be arranged so that withdrawal can be made from three or more levels in the tank. A positive unvalved vented overflow shall be provided. Both primary and secondary digesters should be equipped with supernating lines, so that during emergencies the primary digester can be operated as a single stage process.

5.7.4.11.2.2. Supernatant Selector

A fixed screen supernatant selector or similar type device shall be limited for use in an unmixed secondary digestion unit.

If a supernatant selector is provided, provisions shall be made for at least one other draw-off level located in the supernatant zone of the tank, in addition to the unvalved emergency supernatant draw-off pipe. High-pressure backwash facilities shall be provided.

5.7.4.11.2.3. Withdrawal Selection

On fixed cover tanks the supernatant withdrawal level should preferably be selected by means of interchangeable extensions at the discharge end of the piping.

5.7.4.11.3. Sampling

Provision should be made for sampling at each supernatant draw-off level. Sampling pipes should be at least 40 mm in diameter and should terminate at a suitably sized sampling sink or basin.

5.7.4.11.4. Alternate Supernatant Disposal

An alternate disposal method for the supernatant liquor such as a lagoon, an additional sand bed or hauling from the plant site should be provided for use in case supernatant is not suitable or other conditions make it advisable not to return it to the plant. Consideration should be given to supernatant conditioning where appropriate in relation to its effect on plant performance and effluent quality.

5.7.4.12. Sludge Sampling Requirements

An adequate number of sampling pipes at proper locations should enable the operator to assess the quality of the contents and to know how much sludge is in the digesters. The following requirements shall govern the design:

1. To avoid clogging, sludge sampling pipes should be at least 75 mm in diameter;

2. Provision should be made for the connection of a water source of adequate pressure to these pipes for back flushing when the need arises; and

3. There shall be at least three sampling pipes each separately valved for the primary digesters and four for the secondary digesters.

5.7.5. Aerobic Sludge Digestion

Aerobic digestion is accomplished in single or multiple tanks, designed to provide effective air mixing, reduction of the organic matter, supernatant separation and sludge concentration under controlled conditions.

5.7.5.1. Applicability

Aerobic digestion is considered suitable for secondary sludge or a combination of primary and secondary sludge. Table 5.11 presents the advantages and disadvantages in the use of aerobic sludge digestion.

Table 5.11
Advantages and Disadvantages of Aerobic Sludge Digestion

Advantages	Disadvantages
Low initial cost, especially for small plants	High energy costs
Supernantant less objectionable than anaerobic	Generally lower VSS destruction than anaerobic
Broad applicability	Potential for pathogen spread through aerosol drift
If properly designed, does not generate nuisance odours	Sludge is typically difficult to dewater by mechanical means
Reduces total sludge mass	Cold temperatures adversely affect performance

5.7.5.2. Field Data

Wherever possible, such as in the case of plant expansions, actual sludge quantity data should be considered for digester design. Often, due to errors introduced by poor sampling techniques, inaccurate flow measurements or unmeasured sludge flow streams, the sludge data from existing plants may be unsuitable for use in design. Before sludge data is used for design, it should be assessed for its accuracy.

5.7.5.3. Multiple Units

Multiple digestion units capable of independent operation are desirable and shall be provided in all plants where the design average flow exceeds 455 m^3/d. All plants not having multiple units shall provide alternate sludge handling and disposal methods.

5.7.5.4. Pretreatment

Thickening of sludge is recommended prior to aerobic digestion.

5.7.5.5. Design Considerations
Factors, which should be considered when designing aerobic digesters, include:
1. Type of sludge to be digested;
2. Ultimate method of disposal;
3. Required winter storage;
4. Digester pH;
5. Sludge temperature; and
6. Raw sludge qualities.

5.7.5.6. Solids Retention Time
Where land disposal of digested sludge is practised, a minimum solids retention time of 45 days is required. If local conditions require a more stable sludge, a sludge age of 90 days shall be necessary. To produce a completely stable sludge, a sludge age in excess of 120 days is required.

5.7.5.7. Hydraulic Retention Time
The minimum required hydraulic retention time (HRT) for aerobic digesters provided with pre-thickening facilities are as listed in Table 5.12:

Table 5.12
Minimum HRT

Minimum HRT (days)	Type of Sludge
25	Waste Activated Sludge Only
25	Trickling Filter Sludge Only
30	Primary Plus Secondary Sludge

The more critical of the two guidelines, solids retention time and hydraulic retention time, shall govern the design.

5.7.5.8. Tank Design
5.7.5.8.1. Tank Capacity
The determination of tank capacities shall be based on rational calculations, including factors such as quantity of sludge produced, sludge characteristics, time of aeration and sludge temperature.

Calculations shall be submitted to justify the basis of design. When such calculations are not based on the above factors, the minimum combined digestion tank capacity shall be based on the following:
1. Volatile solids loading shall not exceed 1.60 kg/m^{3}d in the digestion units. Lower loading rates may be necessary depending on temperature, type of sludge and other factors.

2. If a total of 45 days sludge age is all that is provided, it is suggested that ⅔ of the total digester volume be in the first tank and ⅓ be in the second tank. Actual storage requirements will depend upon the ultimate disposal operation. Any minor additional storage requirements may be made up in the second stage digester, but if major additional storage volumes are required, separate on-site or off-site sludge storage facilities should be considered to avoid the power requirements associated with aerating greatly oversized aerobic digesters.

5.7.5.8.2. Air and Mixing Requirements
Aerobic sludge digestion tanks shall be designed for effective mixing by satisfactory aeration equipment. Sufficient air shall be provided to keep the solids in suspension and maintain dissolved oxygen from 1 to 2 mg/L. A minimum mixing and air requirement of 0.85 $L/s/m^3$ of tank volume shall be provided with the largest blower out of service. If diffusers are used, the non-clog type is recommended and they should be designed to permit continuity of service. If mechanical aerators are utilized, at least two turbine aerators per tank shall be provided. Use of mechanical equipment is discouraged where freezing temperatures are normally expected.

Air supply to each tank should be separately valved to allow aeration shutdown in either tank.

5.7.5.8.3. Tank Configuration
Aerobic digesters are generally open tanks. The tankage should be of common wall construction or earthen-bermed to minimize heat loss. Tank depths shall be between 3.5 to 4.5 m; tanks and piping should be designed to permit sludge addition, sludge withdrawal, and supernatant decanting from various depths. Freeboard depths of at least 0.9 to 1.2 m should be provided to account for excessive foam levels. Floor slopes of 1:12 to 3:12 should be provided.

5.7.5.8.4. Supernatant Separation and Scum and Grease Removal
Facilities shall be provided for effective separation or decanting of supernatant. Separate facilities are recommended, however, supernatant separation may be accomplished in the digestion tank provided additional volume is provided. The supernatant draw-off unit shall be designed to prevent recycle of scum and grease back to plant process units. Provision should be made to withdraw supernatant from multiple levels of the supernatant withdrawal zone.

Facilities shall be provided for the effective collection of scum and grease from the aerobic digester for final disposal and to prevent its recycle back to the plant process and to prevent long term accumulation and potential discharge in the effluent.

5.7.5.9. High Level Emergency Overflow
An unvalved high level overflow and any necessary piping shall be provided to return digester overflow back to the head of the plant or to the aeration process in case of accidental overfilling. Design considerations related to the digester overflow shall include waste sludge rate and duration during the period the plant is unattended, potential effect on plant process units, discharge location of the emergency overflow, and potential discharge or suspended solids in the plant effluent.

5.7.5.10. Mixing Tanks and Equipment

Mixing tanks may be designed to operate as either a batch or continuous flow process. A minimum of two tanks shall be provided of adequate size to provide a minimum 2 hours contact time in each tank. The following items shall be considered in determining the number and size of tanks:

1. Peak sludge flow rates;
2. Storage between batches;
3. Dewatering or thickening performed in tanks;
4. Repeating sludge treatment due to pH decay of stored sludge;
5. Sludge thickening prior to sludge treatment; and
6. Type of mixing device used and associated maintenance or repair requirements.

Mixing equipment shall be designed to provide vigorous agitation within the mixing tank, maintain solids in suspension and provide for a homogeneous mixture of the sludge solids and alkaline material. Mixing may be accomplished either by diffused air or mechanical mixers. If diffused aeration is used, an air supply of 0.85 L/m^3s of mixing tank volume shall be provided with the largest blower out of service. When diffusers are used, the non-clog type is recommended, and they should be designed to permit continuity of service. If mechanical mixers are used, the impellers shall be designed to minimize fouling with debris in the sludge and consideration shall be made to provide continuity of service during freezing weather conditions.

5.7.5.11. Chemical Feed and Storage Equipment

Alkaline material is caustic in nature and can cause eye and tissue injury. Equipment for handling or storing alkaline material shall be designed for adequate operator safety. Storage, slaking, and feed equipment should be sealed as airtight as practical to prevent contact of alkaline material with atmospheric carbon dioxide and water vapour and to prevent the escape of dust material. All equipment and associated transfer lines or piping shall be accessible for cleaning.

5.7.5.11.1. Feed and Slaking Equipment

The design of the feeding equipment shall be determined by the treatment plant size, type of alkaline material used, slaking required, and operator requirements. Equipment may be either of batch or automated type. Automated feeders may be of the volumetric or gravimetric type depending on accuracy, reliability, and maintenance requirements. Manually operated batch slaking of quicklime (CaO) should be avoided unless adequate protective clothing and equipment are provided. At small plants, used of hydrated lime [$Ca(OH)_2$] is recommended over quicklime due to safety and labour-saving reasons. Feed and slaking equipment shall be sized to handle a minimum of 150% of the peak sludge flow rate including sludge that may need to be retreated due to pH decay. Duplicate units shall be provided.

5.7.5.11.2. Chemical Storage Facilities

Alkaline materials may be delivered either in bag or bulk form depending upon the amount of material used. Material delivered in bags must be stored indoors and elevated above floor level. Bags should be of the multi-wall moisture-proof type. Dry bulk storage containers must be as airtight as practical and shall contain a mechanical agitation mechanism. Storage facilities shall be sized to provide a minimum of a 30-day supply.

5.7.5.12. Sludge Storage

The design shall incorporate considerations for the storage of high pH stabilized sludge, as per the following sub-sections.

5.7.5.12.1. Liquid Sludge

Liquid high pH stabilized sludge shall not be stored in a lagoon. Said sludge shall be stored in a tank or vessel equipped with rapid sludge withdrawal mechanisms for sludge disposal or re-treatment. Provisions shall be made for adding alkaline material in the storage tank. Mixing equipment in accordance with Section 5.7.5.10 shall also be provided in all storage tanks.

5.7.5.12.2. Dewatered Sludge

On-site storage of dewatered high pH stabilized sludge should be limited to 30 days. Provisions for rapid re-treatment or disposal of dewatered sludge stored on-site shall also be made in case of sludge pH decay.

5.7.5.12.3. Off-Site Storage

There shall be no off-site storage of high pH stabilized sludge unless specifically permitted the DOEC.

5.7.5.13. Sludge Disposal

Immediate sludge disposal methods and options are recommended to be utilized in order to reduce the sludge inventory on the treatment plant site and amount of sludge that may need to be retreated to prevent odours if sludge pH decay occurs. If the land application disposal option is utilized for high pH stabilized sludge, said sludge must be incorporated into the soil during the same day of delivery to the site.

5.7.6. Sludge Dewatering

Sludge dewatering will often be required at sewage treatment plants prior to ultimate disposal of sludges. Since the processes differ significantly in their ability to reduce the water content of sludges, the ultimate sludge disposal method will generally have a major influence on the dewatering method most suitable for a particular sewage treatment plant. Also of influence will be the characteristics of the sludge requiring dewatering, that is, whether the sludge is raw or digested, whether the sludge contains waste activated sludge, or whether the sludge has been previously thickened. With raw sludge, the freshness of the sludge will have a significant effect on dewatering performance (septic sludge will be more difficult to dewater than fresh raw sludge).

As with thickening systems, dewatering facilities may require sludge pre-treatment in the form of sludge grinding to avoid plugging pumps, lines and plugging or damaging dewatering equipment. Also, adequate ventilation equipment will be required in buildings housing dewatering equipment. In evaluating dewatering system alternatives, the designer must consider the capital and operating costs, including labour, parts, chemicals and energy, for each alternative as well as for the effects which each alternative will have on the sewage treatment and subsequent sludge handling and ultimate sludge disposal operations.

In considering the need for sludge dewatering facilities, the designer should evaluate the economics of the overall treatment processes, with and without facilities for sludge water content reduction. This evaluation should consider both capital and operating costs of the various plant components and sludge disposal operations affected.

Wherever possible, pilot plant and/or bench-scale data should be used for the design of dewatering facilities. With new plants, this may not always be possible and, in such cases, empirical design parameters must be used. The following subsections outline the normal ranges for the design parameters of such equipment. For calculating dewatering design sludge handling needs, a rational basis of design for sludge production from sludge stabilization processes shall be developed and provided to the regulatory agencies for approval on a case-by case basis.

5.7.6.1. Dewatering Process Compatibility with Subsequent Treatment or Disposal Techniques

Table 5.13 outlines the relationship of dewatering to other sludge treatment processes.

Table 5.13
Relationship of Dewatering to Other Sludge Treatment Processes
For Typical Municipal Sludge

Method	Pretreatment Normally Provided		Normal Use of Dewatering Cake			
	Thickening	Conditioning	Landfill	Land Spread	Heat Drying	Incineration
Rotary Vacuum Filter	Yes	Yes	Yes	Yes	Yes	Yes
Centrifuge (solid bowl)	Yes	Yes	Yes	Yes	Yes	Yes
Centrifuge (basket)	Variable	Variable	Yes	Yes	No	No
Drying Beds	Variable	Not Usually	Yes	Yes	No	No
Lagoons	No	No	Yes	Yes	No	No
Filter Presses	Yes	Yes	Yes	Variable	Not Usually	Yes
Horizontal Belt Filters	Yes	Yes	Yes	Yes	Yes	Yes

5.7.6.2. Sludge Drying Beds

5.7.6.2.1. Pretreatment

Sludge shall be pretreated before being air-dried by either one of the following methods:

1. Anaerobic digesters;
2. Aerobic digesters with provision to thicken;
3. Digestion in aeration tanks of extended aeration plants (with long sludge age, greater than about 20 days) preferably with provision to thicken using thickeners, lagoons, or by other means; or
4. Well designed and maintained oxidation ditches with sludge age longer than about 20 days (preferably after thickening).

5.7.6.2.2. Chemical Conditioning

The dewatering characteristics can be considerably improved by chemical conditioning of sludge prior to treatment in beds. Since sludge conditioning can reduce the required drying time to one-third or less, of the unconditioned drying time, provision should be made for the addition of conditioning chemicals, usually polymers.

5.7.6.2.3. Design Criteria

5.7.6.2.3.1. Factors Influencing Design

The design and operation of sludge drying beds depend on the following factors:

1. Climate in the area;
2. Sludge characteristics;
3. Pre-treatment (such as conditioning, thickening, etc.); and
4. Sub-soil permeability.

5.7.6.2.3.2. Bed Area

Consideration should be given to the following when calculating the bed area:

1. The volume of wet sludge produced by existing and proposed processes;

2. Depth of wet sludge drawn to the drying beds. For design calculation purposes a maximum depth of 200 mm shall be utilized. For operational purposes, the depth of sludge placed on the drying bed may increase or decrease from the design depth based on the percent solids content and type of digestion utilized;

3. Total digester volume and other wet sludge storage facilities;

4. Degree of sludge thickening provided after digestion;

5. The maximum drawing depth of sludge, which can be, removed from the digester or other sludge storage facilities without causing process or structural problems;

6. The time required on the bed to produce a removable cake. Adequate provision shall be made for sludge dewatering and/or sludge disposal facilities for those periods of time during which outside drying of sludge on beds in hindered by weather; and

7. Capacities of auxiliary dewatering facilities. Sludge drying beds may be designed from basic principles, laboratory tests, and/or pilot plant field studies. Calculations must be presented to the DOEC supporting any design based on the above methods. In the absence of such calculations the minimum sludge drying bed shall be based on the criteria presented in Table 5.14.

Table 5.14
Sludge Drying Bed Areas

Type of Wastewater Treatment	Area (m^3/capita)		
	Open Beds	Covered Beds	Combination of Open and Covered Beds
Primary Plants (no secondary treatment)	0.12	0.10	0.10
Activated Sludge (no primary treatment)	0.16	0.13	0.13
Primary and Activated Sludge	0.20	0.16	0.16

The area of the bed may be reduced by up to 50% if it is to be used solely as a back-up dewatering unit. An increase of bed area by 25% is recommended for paved beds.

5.7.6.2.3.3. Percolation Type Beds

1. **Pond Bottom** - The bottom of the cell should be of impervious material such as clay or asphalt.

2. **Underdrains** - Underdrains should at least 100 mm in diameter laid with open joints. Perforated pipe may also be used. Underdrains should be spaced 2.5 to 3.0 m apart, with a slope of 1.0 %, or more. Underdrains should discharge back to the secondary treatment section of the sewage treatment plant. Various pipe materials may be selected provided the material is of suitable strength and corrosion resistant.

3. **Gravel** - The lower course of gravel around the underdrains should be properly graded and should be 300 mm in depth, extending at least 150 mm above the top of the underdrains. It is desirable to place this in two or more layers. The top layer, of at least 75 mm in depth, should consist of gravel 3 mm to 6 mm in size. The gravel should be graded from 25 mm on the bottom to 3 mm on the top.

4. **Sand** - The top course should consist of 250 to 450 mm of clean coarse sand. The effective size should range from 0.3 to 1.2 mm with uniformity co-efficient of less than 5.0. The finished sand surface should be level.

5. **Additional Dewatering Provisions** - Consideration shall be given for providing a means of decanting supernatant of sludge placed on the sludge drying beds. More effective decanting of supernatant may be accomplished with polymer treatment of sludge.

5.7.6.2.3.4. Impervious Type Beds

Paved drying beds should be designed with consideration for space requirements to operate mechanical equipment for removing the dried sludge.

5.7.6.2.3.5. Location

Depending on prevailing wind directions, a minimum distance of 100 to 150 m shall be kept from open sludge drying beds and dwellings. However, the minimum maybe reduced to 60 m to 80 m for enclosed beds. The selected location for open beds shall be at least 30 m from public roads and 25 m for enclosed beds. The plant owner may be required to spray deodorants and odour masking chemicals whenever there are complaints from the population in the neighbourhood.

5.7.6.2.3.6. Winter Storage

Alternative methods of disposal should be arranged for the non-drying season, which may start as early as October (or November) and end in April (or March).

5.7.6.2.3.7. Dimensions

The bed size generally should be 4.5 to 7.5 m wide with the length selected to satisfy desired bed loading volume.

5.7.6.2.3.8. Depth of Sludge

The sludge dosing depth shall generally be 200 to 300 mm for warm weather operating modes; for winter freeze drying depths of 1 to 3 m can be used depending upon the number of degree days in winter.

5.7.6.2.3.9. Number of Beds

Three beds are desirable for increased flexibility of operation. Not less than two beds shall be provided.

5.7.6.2.3.10. Walls

Walls should be watertight and extend 400 to 500 mm above and at least 150 mm below the surface. Outer walls should be extended at least 100 mm above the outside grade elevation to prevent soil from washing on to the beds.

5.7.6.2.3.11. Sludge Influent

The sludge pipe to the beds should terminate at least 300 mm above the surface and be so arranged that it will drain. Concrete splash plates for percolation type beds should be provided at sludge discharge points. One inlet pipe per cell should be provided.

5.7.6.2.3.12. Sludge Removal

Each bed shall be constructed so as to be readily and completely accessible to mechanical cleaning equipment. Concrete runways spaced to accommodate mechanical equipment shall be provided. Special attention should be given to assure adequate access to the areas adjacent to the sidewalls. Entrance ramps down to the level of the sand bed shall be provided. These ramps should be high enough to eliminate the need for an entrance end wall for the sludge bed.

Atlantic Canada climatological conditions may permit 3 or 4 cycles (consisting of filling the open bed with digested sludge, drying and emptying) during the drying season. However, the number of cycles may be increased to approximately 10 with covered beds. These values are tentative and subject to revision after field observations.

5.7.6.2.3.13. Covered Beds
Consideration should be given to the design and use of covered sludge drying beds.

5.7.6.3. Sludge Lagoons
Sludge drying lagoons may be used as a substitute for drying beds for the dewatering of digested sludge. Lagoons are not suitable for dewatering untreated sludge, limed sludge, or sludge with a high strength supernatant because of their odour and nuisance potential. The performance of lagoons, like that of drying beds, is affected by climate; precipitation and low temperatures inhibit dewatering. Lagoons are most applicable in areas with high evaporation rates.

Sludge lagoons may also be used as temporary sludge storage facilities, when spreading on agricultural land cannot be carried out due to such factors as wet ground, frozen ground or snow cover.

Sludge lagoons as a means of dewatering digested sludge will be permitted only upon proof that the character of the digested sludge and the design mode of operation are such that offensive odours will not result. Where sludge lagoons are permitted, adequate provisions shall be made for other sludge dewatering facilities or sludge disposal in the event of upset or failure of the sludge digestion process.

5.7.6.3.1. Design Considerations
The design and location of sludge lagoons must take into consideration many factors, including the following:
1. Possible nuisances – odours, appearances, mosquitoes;
2. Design – number, size, shape and depth;
3. Loading factors – solids concentration of digested sludge, loading rates;
4. Soil conditions – permeability of soil, need for liner, stability of berm slopes, etc.;
5. Groundwater conditions – elevation of maximum groundwater, level, direction of groundwater movement, location of wells in the area;
6. Sludge and supernatant removal – volumes, concentrations, methods of removal, method of supernatant treatment and final sludge disposal; and
7. Climatic effects – evaporation, rainfall, freezing, snowfall, temperature, solar radiation.

5.7.6.3.2. Pre-Treatment
Pre-treatment requirements for sludge lagoons are the same as those for sludge drying beds.

5.7.6.3.3. Soil and Groundwater Conditions
The soil must be reasonably porous and the bottom of the lagoons must be at least 1.2 m above the maximum ground water table. Surrounding areas shall be graded to prevent surface water entering the lagoon. In some critical instances, the DOEC may require a lagoon to be lined with plastic or rubber material.

5.7.6.3.4. Depth
Lagoons should be at least 1 m in depth while maintaining a minimum of 0.6 m of freeboard.

5.7.6.3.5. Seal
Adequate provisions shall be made to seal the sludge lagoon bottom and embankments in accordance with the requirements of Section 5.9.6.2 to prevent leaching into adjacent soils or ground water.

5.7.6.3.6. Area
The area required will depend on local climatic conditions. Not less than two lagoons should be provided.

5.7.6.3.7. Location
Consideration shall be given to prevent pollution of ground and surface water. Adequate solation shall be provided to avoid nuisance production.

5.7.6.3.8. Cycle Time and Sludge Removal
The cycle time for lagoons varies from several months to several years. Typically, sludge is pumped to the lagoon for 18 months and then the lagoon is rested for 6 months. Sludge is removed mechanically, usually at a moisture content of about 70 %.

5.7.6.4. Mechanical Dewatering Facilities
Provisions shall be made to maintain sufficient continuity of service so that sludge may be dewatered without accumulation beyond storage capacity. If it is proposed to dewater the sludge by mechanical methods such as rotary vacuum filters, centrifuges, filter presses or belt filters, a detailed description of the process and design data shall accompany the plans. Unless standby facilities are available, adequate storage facilities shall be provided. The storage capacity should be sufficient to handle at least 4 days of sludge production volume.

5.7.6.4.1. Performance of Mechanical Dewatering Methods
Table 5.15 outlines the solids capture, solids concentrations normally achieved and energy requirements for various mechanical dewatering methods.

Table 5.15
Sludge Dewatering Methods and Performance with Various Sludge Types

Dewatering Method	Solids Capture (%)	Solids Concentrations Normally Achieved (1)	Median Energy Required (MJ/Dry Tonne) (2)
Vacuum Filter	90 – 95	Raw primary + was (10-25%) Digested primary + was (15-20%) Was (8-12%)	1080
Filter Press	90 – 95	Raw primary + was (30-50%) Digested primary + was (35-50%) Was (25-50%)	360
Centrifuge (solid bowl)	95 – 99	Raw or Digested primary + was (15-25%) Was (12-15%)	360
Belt Filter	85 - 95	Raw or Digested primary + was (14-25%) Was (10-15%)	130

1. Including conditioning chemicals, if required.
2. MJ/Dry Tonne – denotes mega joules per dry tonne of sludge throughout.

5.7.6.4.2. Number of Units
With sludge dewatering equipment, multiple units will generally be required unless satisfactory sludge storage facilities or alternate sludge disposal methods are available for use during periods of equipment repair. Often the need for full standby units will be unnecessary if the remaining duty units can be operated for additional shifts in the event of equipment breakdown. There shall be a back up pump and filtrate pump installed for each vacuum filter.

5.7.6.4.3. Ventilation
Adequate facilities shall be provided for ventilation of the dewatering area. The exhaust air should be properly conditioned to avoid odour nuisance.

5.7.6.4.4. Chemical Handling Enclosures
Lime-mixing facilities should be completely enclosed to prevent the escape of lime dust. Chemical handling equipment should be automated to eliminate the manual lifting requirement.

5.7.6.4.5. Drainage and Filtrate Disposal
Drainage from beds or filtrate from dewatering units shall be returned to the sewage treatment process at appropriate points.

5.7.6.4.6. Other Dewatering Facilities
If it is proposed to dewater sludge by mechanical means, other than those outlined below, a detailed description of the process and design shall accompany the plans.

5.7.6.4.7. Vacuum Filters

Of primary importance with vacuum filters is the solids concentration of sludge fed to the units. With all other operating variables remaining constant, increases in filtration rates vary in direct proportion to feed solids. Sludge thickening prior to vacuum filters is therefore extremely important. Higher concentrations in the sludge feed also result in lower filtrate solids.

Vacuum filtration systems should be designed in accordance with the following parameters:

1. Sludge feed pumps:
   a) Variable capacity;

2. Vacuum pumps:
   a) Generally one per machine with capacity of 10 L/m^2s at 65 kPa or more, vacuum;

3. Vacuum receiver:
   a) Generally one per machine;
   b) Maximum air velocity 0.8 to 1.5 m/s;
   c) Air retention time 2-3 minutes;
   d) Filtrate retention time 4-5 minutes;
   e) All lines shall slope downward to the receiver from the vacuum filter;

4. Filtrate pumps:
   a) Generally self-priming centrifugal;
   b) Suction capacity greater than vacuum pump, 65 to 85 kPa vacuum;
   c) With flooded pump suctions;
   d) With check valve on the discharge side to minimize air leakage into the system;
   e) Pumps must be sized for the maximum expected sludge drainage rates (usually produced by polymers);

5. Sludge flocculation tank:
   a) Constructed of corrosion-resistant materials; and
   b) With slow speed variable drive mixer, detention time 2-4 minutes with ferric and lime (with polymers shorter time may be used);

6. Wash water:
   a) Filtered final effluent generally used;

7. Sludge measurement:
   a) Should be provided unless measured elsewhere in plant;

8. Solids loading rate:
   a) 7-14 g/m^2s for raw primary;
   b) 2.75-7 g/m^2s for raw primary + WAS; 4-7 g/m^2s for digested primary + WAS; and
   c) Not considered practical for use with WAS alone.

5.7.6.4.8. Filter Presses

As with vacuum filters, the capacity of filter presses is greatly affected by the initial solids concentration. With low feed solids, chemical requirements increase significantly. Sludge, thickening should therefore be considered as a pre-treatment step. Filter press systems should be designed in accordance with the following guidelines:

1. Sludge conditioning tank:
   a) Detention time maximum 20 minutes at peak pumpage rate;

2. Feed Pumps:
   a) Variable capacity to allow pressures to be increased gradually, without underfeeding or overfeeding sludge;
   b) Pumps should be of a type to minimize floc shear;
   c) Pumps must deliver high volume at low head initially and low volume at high head during latter part of cycle; and
   d) Ram or piston pumps, progressing cavity pumps or double diaphragm pumps are generally used;

3. Cake handling:
   a) Filter press must be elevated above cake conveyance system to allow free fall; and
   b) Cake can be discharged directly to trucks, into dumper boxes, or onto conveyors (usually cable cake breakers may be needed);

4. Cycle times:
   a) 1.5 to 6 hours (normally 1.5 to 3 hours); and

5. Operating pressures:
   a) Usually 700 to 1400 kPa, but may be as high as 1750 kPa. The operating pressure shall not exceed 1000 to 1050 kPa, if polymer is applied as the conditioning agent.

5.7.6.4.9. Solid Bowl Centrifuges

Bowl length/diameter ratios of 2.5 to 4.0 should be provided to ensure adequate settling time and surface area. Bowl angles must be kept shallow. The bowl flow pattern can be either counter-current or concurrent. Pool depth can be varied by adjustable weirs. Conveyor design and speed will affect the efficiency of solids removal. Differential speed must be kept low enough to minimize turbulence and internal wear yet high enough to provide sufficient solids handling capacity.

For most wastewater sludges, the capacity of the centrifuge will be limited by the clarification capacity (hydraulic capacity) and therefore the solids concentration. Increasing the feed solids will increase the solids handling capacity. Thickening should, therefore, be considered as a pre-treatment operation.

Since temperature affects the viscosity of sludges, if the temperatures will vary appreciably (as with aerobic digestion), the required centrifuge capacity should be determined for the lowest temperature expected. Other general design guidelines for solid bowl centrifuges are as follows:

1. Feed pump:
   a) Sludge feed should be continuous;
   b) Pumps should be variable flow type;
   c) One pump should be provided per centrifuge for multiple centrifuge systems; and
   d) Chemical dosage should vary with the pumpage rate;

2. Sludge pre-treatment:
   a) Depending upon the sewage treatment process, grit removal, screening or maceration may be required for the feed sludge stream;

3. Solids Capture:
   a) 85 to 95% is generally desirable;

4. Machine materials:
   a) Generally carbon steel or stainless steel; and
   b) Parts subject to wear should be protected with hard facing materials such as a tungsten carbide material;

5. Machine foundations:
   a) Foundations must be capable of absorbing the vibratory loads;

6. Provision for maintenance:
   a) Sufficient space must be provided around the machine(s) to permit disassembly;
   b) An overhead hoist should be provided;
   c) Hot and cold water supplies will be needed to permit flushing out the machine; and
   d) Drainage facilities will be necessary to handle wash water.

5.7.6.4.10. Belt Filter Presses

Most types of wastewater sludges can be dewatered with belt filter presses and the results achieved are generally superior to those of vacuum filters. Chemical conditioning is generally accomplished with polymer addition. Solids handling capabilities are likely to range from 50 g/m·s (based on belt width) for excess activated sludge to 330 g/m·s for primary sludge.

5.7.7. Sludge Pumps and Piping

5.7.7.1. Sludge Pumps

5.7.7.1.1. General Sludge Pumping Requirements

Table 5.16 outlines general sludge pumping requirements for various sludge types.

Table 5.16
Sludge Pumping Requirements for Various Sludge Types

Sludge Source	**Slurry** (% total solids)	**Static Head** (m)	**TDH** (m)	**Abrasive Service**	**Duty**
Pre-treatment - Grit	0.5 – 10	0 – 1.5 (gravity)	1.5 – 3	Yes – High	Heavy
Primary Sedimentation Unthickened Thickened	0.2 – 2 4 – 10	3 – 12 3 – 12	10 – 200 12 – 25	Yes Yes	Medium Heavy
Secondary Sedimentation (for re-circulation)	0.5 – 2	1 – 2	3 – 4.5	No	Light
Secondary Sedimentation (for thickening)	0.5 – 2	1.2 – 2.4	3 – 4.5	No	Light
Thickener Underflow	5 – 10 5 – 10	6 – 12 60 – 120**	25 – 45 75 – 170	Yes/No* Yes/No*	Heavy Very Heavy
Digester Re-circulation Underflow	3 – 10 3 – 10	0 – 1.5 0 – 6	2.4- 3.6 15 – 30	No Yes/No*	Medium Very Heavy
Chemically Produced Sludges: Alum/Ferric – primary Lime – primary Lime – secondary	 0.5 – 310 1 – 6 2 – 15	 3 – 12 3 – 12 3 – 12	 9 – 20 9 – 25 9 – 25	 No No No	 Light Medium Medium
Incinerator Slurries	0.5 - 10	0 - 15	6 – 30	Yes - High	Heavy

Note: * Depends on degritting efficiency
** High pressure for heat treatment

5.7.7.1.2. Capacity

Pump capacities should be adequate but not excessive. Provision for varying pump capacity is desirable.

5.7.7.1.3. Duplicate Units

Duplicate units shall be provided where failure of one unit would seriously hamper plant operation.

5.7.7.1.4. Type

Plunger pumps, screw feed pumps or other types of pumps with demonstrated solids handling capability should be provided for handling raw sludge. Where centrifugal pumps are used, a parallel positive displacement pump should be provided as an alternate to pump heavy sludge concentrations, such as primary or thickened sludge, that may exceed the pumping head of the centrifugal pump.

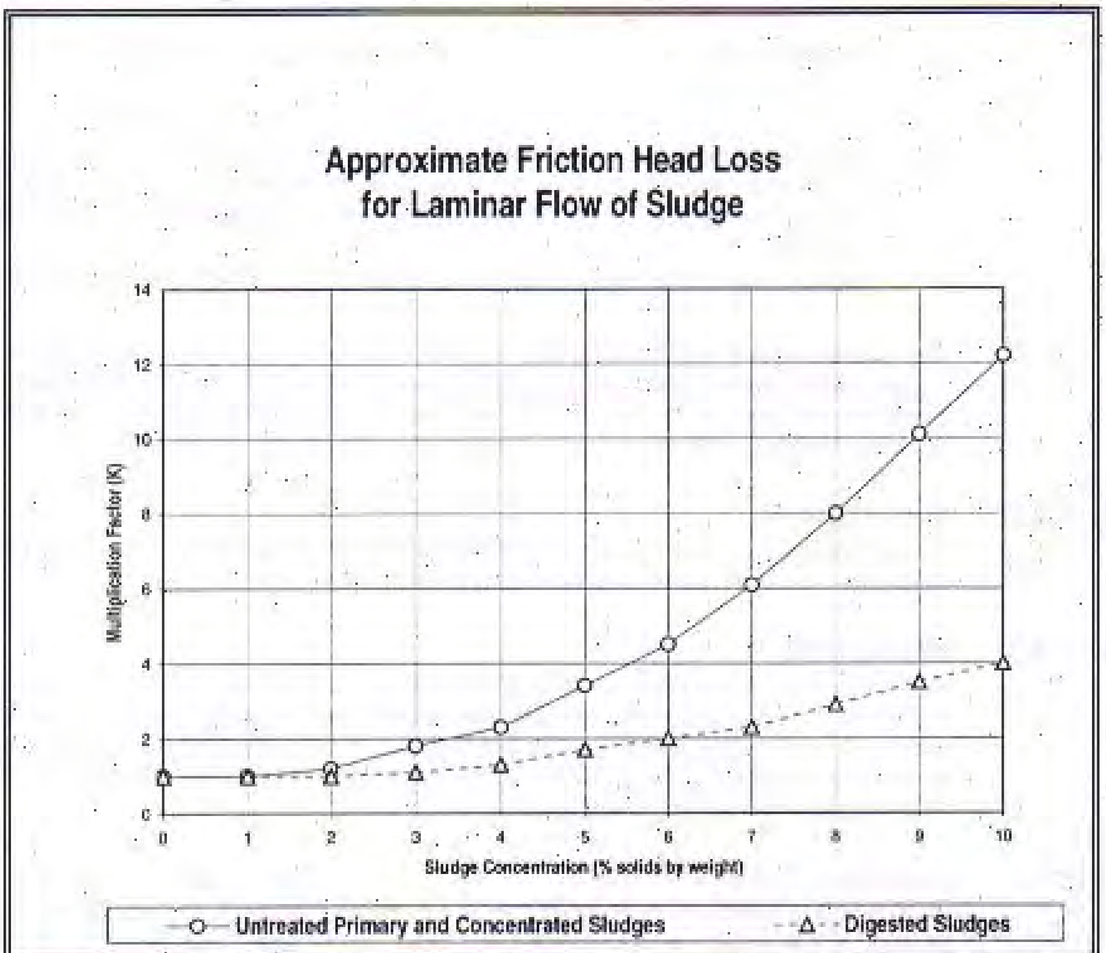
5.7.7.1.5. Minimum Head

A minimum positive head of 600 mm shall be provided at the suction side of centrifugal type pumps and is desirable for all types of sludge pumps. Maximum suction lifts should not exceed 3 m for plunger pumps.

5.7.7.1.6. Head Loss

Figure 5.2 shows the multiplication factor to apply to the friction losses for turbulent flow for clean water to calculate the friction losses for untreated primary and concentrated sludges and digested sludge. Use of Figure 5.2 will often provide sufficiently accurate results for design, especially at solids concentrations below 3 %. However, as pipe length, percent total solids and percent volatile solids increase, more elaborate methods may have to be used to calculate the friction losses with sufficient accuracy.

Figure 5.2



NOTES: 1. Multiply loss with clean water by K to estimate friction loss under laminar conditions (see text).

2. The information on this figure has been extracted from EPA 625/1-79-011 "Process Design Manual for Sludge Treatment and Disposal:, September 1979.

5.7.7.1.7. Sampling Facilities
Unless sludge-sampling facilities are otherwise provided, quick closing sampling valves shall be installed at the sludge pumps. The size of valve and piping should be at least 40 mm and terminate at a suitable sized sampling sink or floor drain.

5.7.7.2. Sludge Piping
5.7.7.2.1. Size and Head
Sludge withdrawal piping should have a minimum diameter of 200 mm for gravity withdrawal and 150 mm for pump suction and discharge lines. Where withdrawal is by gravity, the available head on the discharge pipe should be adequate to provide at least 1.0 m/s velocity. With sludge pumpage velocities of 0.9 to 1.5 m/s should be developed. For heavier sludges and grease, velocities of 1.5 to 2.4 m/s are needed.

5.7.7.2.2. Slope
Gravity piping should be laid on uniform grade and alignment. The slope on gravity discharge piping should not be less than 3 percent. Provisions should be made for draining and flushing discharge lines.

5.7.7.2.3. Supports
Special consideration should be given to the corrosion resistance and continuing stability of supporting systems for piping located inside the digestion tank.

5.8. Biological Treatment

5.8.1. Activated Sludge

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

5.8.1.2. Process Selection
The activated sludge process and its several modifications may be employed to accomplish varied degrees of removal of suspended solids and reduction of carbonaceous and/or nitrogenous oxygen demand. Choice of the process most applicable will be influenced by the degree and consistency of treatment required, type of waste to be treated, proposed plant size, anticipated degree of operation and maintenance, and operating and capital costs. All designs should provide for flexibility in operation. Plants over 4500 m^3/d should be designed to facilitate easy conversion to various operation modes, if feasible.

The design must be based on experience at other facilities. Continuity and reliability of treatment equal to that of the continuous flow through modes of the activated sludge process shall be provided. The DOEC shall be contacted for design guidance and criteria where such systems are being considered.

5.8.1.3. Energy Requirements

This process requires major energy usage to meet aeration demands. Energy costs in relation to critical water quality conditions must be carefully evaluated. Capability of energy usage phase-down while still maintaining process viability, both under normal and emergency energy availability conditions, must be included in the activated sludge design.

5.8.1.4. Winter Protection

Protection against freezing shall be provided to ensure continuity of operation and performance.

5.8.1.5. Pretreatment

Where primary settling tanks are not used, effective removal or exclusion of grit, debris, excessive oil or grease, and comminution or screening of solids shall be accomplished prior to the activated sludge process.

Where primary settling is used, provision shall be made for discharging raw sewage directly to the aeration tanks to facilitate plant start-up and operation during the initial stages of the plant's design life.

5.8.1.6. Aeration

5.8.1.6.1. Capacities and Permissible Loadings

The size of the aeration tank for any particular adaptation of the process should be determined by full-scale experience, pilot plant studies, or rational calculations based mainly on food to microorganism ratio and mixed liquor suspended solids levels. Other factors such as size of treatment plant, diurnal load variations, and degree of treatment required should also be considered. In addition, temperature, pH, and reactor-dissolved oxygen should be considered when designing for nitrification.

Calculations should be submitted to justify the basis for design of aeration tank capacity. Calculations using values differing substantially from those in the accompanying table should reference actual operational plants. Mixed liquor suspended solids levels greater than 5000 mg/L may be allowed providing adequate data is submitted showing the aeration and clarification system capable of supporting such levels.

When process design calculations are not submitted, the aeration tank capacities and permissible loadings for several adaptations of the processes shown in Table 5.17 shall be used. These values apply to plants receiving diurnal load ratios of design peak hourly BOD_5 to design average BOD_5 ranging from about 2:1 to 4:1. Thus, the utilization of flow equalization facilities to reduce the diurnal design peak hourly BOD_5 organic load may be considered by the DOEC as justification to approve organic loading rates that exceed those specified in Table 5.17.

Table 5.17
Allowable Aeration Tank Capacity and Loading

Type of Process	Percentage of Average Design Flow		
	Minimum (%)	Normal (%)	Maximum (%)
Plug Flow	25	30	100
Complete Mix	25	30	100
Carbonaceous Stage of Separate Sludge	25		75
Nitrification	25	50	75
Step Aeration	50	100	150
Contact Stabilization	50	100	150
Extended Aeration	50	100	150
Oxidation Ditch	50	50	200
High Rate Nitrification Stage of Separate Stage Nitrification	50		200

5.8.1.6.2. Arrangement of Aeration Tanks

1. Dimensions
   The dimensions of each independent mixed liquor aeration tank or return sludge re-aeration tank shall be such as to maintain effective mixing and utilization of air. Ordinarily, liquid depths should not be less than 3 m or more than 9 m except in special design cases.

2. Short Circuiting
   For very small tanks or tanks with special configuration, the shape of the tank and the installation of aeration equipment should provide for positive control of short-circuiting through the tank.

5.8.1.6.2.1. Number of Units

Total aeration tank volume should be divided among two or more units, capable of independent operation, when required by the DOEC to meet applicable effluent limitations and reliability guidelines.

5.8.1.6.2.2. Inlets, Outlets and Conduits

Inlets and outlets for each aeration tank unit should be suitably equipped with valves, gates, stop plates, weirs, or other devices to permit controlling the flow to any unit and to maintain reasonably constant liquid level. The effluent weir for a horizontally mixed aeration tank system must be easily adjustable by mechanical means and shall be sized based on the design peak instantaneous flow plus the maximum return sludge flow. The hydraulic properties of the system should permit the design peak instantaneous flow to be carried with any single aeration tank unit out of service.

Channels and pipes carrying liquids with solids in suspension should be designed to maintain self-cleansing velocities or should be agitated to keep such solids in suspension at all rates of flow within the design limits. Adequate provisions should be made to drain segments of channels, which are not being used due to alternate flow patterns.

5.8.1.6.2.3. Freeboard

All aeration tanks should have a freeboard of not less than 450 mm. However, if a mechanical surface aerator is used, the freeboard should not be less than 1 m to protect against windblown spray freezing on walkways, etc.

5.8.1.6.3. Aeration Equipment

Oxygen requirements generally depend on maximum diurnal organic loading, degree of treatment, and level of suspended solids concentration to be maintained in the aeration tank mixed liquor. Aeration equipment should be capable of maintaining a minimum of 2.0 mg/L of dissolved oxygen in the mixed liquor at all times and providing thorough mixing of the mixed liquor. In the absence of experimentally determined values, the design oxygen requirements for all activated sludge processes should be 1.1 kg of O_2/kg of peak BOD_5 applied to the aeration tanks with the exception of the extended aeration process, for which the value should be 1.5 to include respiration requirements.

Where nitrification is required or will occur, such as within the extended aeration process, the oxygen requirement for oxidizing ammonia must be added to the above requirement for carbonaceous BOD_5 removal and respiration requirements. The nitrogenous oxygen demand (NOD) should be taken as 4.6 times the diurnal peak hourly TKN content of the influent. In addition, the oxygen demands due to recycle flows - heat treatment supernatant, vacuum filtrate, elutriates, etc. - must be considered due to the high concentrations of BOD_5 and TKN associated with such flows.

Careful consideration should be given to maximizing oxygen utilization per unit power input. Unless flow equalization is provided, the aeration system should be designed to match the diurnal organic load variation while economizing on power input.

5.8.1.6.3.1. Aeration Equipment Selection

Evaluation of aeration equipment alternatives should include the following considerations:

1. Costs – capital, maintenance and operating;
2. Oxygen transfer efficiency;
3. Mixing capabilities;
4. Diffuser clogging problems;
5. Air pre-treatment requirements;
6. Total power requirements;
7. Aerator tip speed of mechanical aerators used with activated sludge systems;
8. Icing problems;
9. Misting problems; and
10. Cooling effects on aeration tank contents.

The size of the aeration tank for any particular adaptation of the process shall be determined by full-scale experience, pilot plant studies, or rational calculations based mainly on food to microorganism ratio and mixed liquor suspended solids levels. Other factors, such as size of treatment plant, diurnal load variations, and degree of treatment required, shall also be considered. In addition, temperature, pH, and reactor-dissolved oxygen shall be considered when designing for nitrification.

5.8.1.6.3.2. Diffused Air Systems

The design of the diffused air system to provide the oxygen requirements shall be done by either of the two methods described below in (1) and (2), augmented as required by consideration of items (3) through (8):

1. Having determined the oxygen requirements per Section 5.8.1.6.3, air requirements for a diffused air system shall be determined by use of any of the well known equations incorporating such factors as:
   a) Tank depth;
   b) Alpha factor of waste;
   c) Beta factor of waste;
   d) Certified aeration device transfer efficiency;
   e) Minimum aeration tank dissolved oxygen concentrations.
   f) Critical wastewater temperature; and
   g) Altitude of plant.

   In the absence of experimentally determined alpha and beta factors, wastewater transfer efficiency should be assumed to be 50% of clean water efficiency for plants treating primarily (90% or greater) domestic wastewater. Treatment plants where the waste contains higher percentage of industrial wastes should use a correspondingly lower percentage of clean water efficiency and shall have calculations submitted to justify such a percentage. The design transfer efficiency should be included in the specifications.

2. Normal air requirements for all activated sludge processes except extended aeration, (assuming equipment capable of transmitting to the mixed liquor the amount of oxygen required in Section 5.8.1.6.3), shall be considered to be 94 m^3 at standard conditions of pressure, temperature and humidity per kilogram of BOD_5 tank loading. For the extended aeration process the value shall be 125 m^3/kg of BOD_5.

3. To the air requirements calculated above shall be added air required for channels, pumps, aerobic digesters, filtrate and supernatant or other air-use demand.

4. The specified capacity of blowers or air compressors, particularly centrifugal blowers, should take into account that the air intake temperature may reach 40^oC or higher and the pressure may be less than normal. The specified capacity of the motor drive should also take into account that the intake air may be -30^oC or less and may require over-sizing of the motor or a means of reducing the rate of air delivery to prevent overheating or damage to the motor.

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

Aeration equipment shall be easily adjustable in increments and shall maintain solids suspension within these limits.

6. Diffuser systems shall be capable of providing for 200% of the design average oxygen demand. The air diffusion piping and diffuser system should be capable of delivering normal air requirements with minimal friction losses. Air piping systems should be designed such that total pressure loss from blower outlet (or silencer outlet where used) to the diffuser inlet does not exceed 3.4 kPa at average operating conditions. The spacing of diffusers shall be in accordance with the oxygen requirements through the length of the channel or tank, and shall be designed to facilitate adjustment of their spacing without major revision to air header piping. All plants employing less than four independent aeration tanks shall be designed to incorporate removable diffusers that can be serviced and/or replaced without dewatering the tank

7. Individual assembly units of diffusers should be equipped with control valves, preferably with indicator markings for throttling, or for complete shutoff. Diffusers in any single assembly shall have substantially uniform pressure loss.

8. Air filters shall be provided in numbers, arrangements, and capacities to furnish at all times an air supply sufficiently free from dust to prevent damage to blowers and clogging of the diffuser system used.

5.8.1.6.3.3. Mechanical Aeration Systems

5.8.1.6.3.3.1. Oxygen Transfer Performance

The mechanism and drive unit shall be designed for the expected conditions in the aeration tank in terms of the power performance. Certified testing shall verify mechanical aerator performance. Refer to applicable provisions of Section 5.8.1.6.3.2. In the absence of specific design information, the oxygen requirements shall be calculated using a transfer rate not to exceed 1.22 kg O_2/kW·hr in clean water under standard test conditions. Design transfer efficiencies shall be included in the specifications.

5.8.1.6.3.3.2. Design Requirements

The design requirements of a mechanical aeration system shall accomplish the following:

1. Maintain a minimum of 2.0 mg/L of dissolved oxygen in the mixed liquor at all times throughout the tank or basin;
2. Maintain all biological solids in suspension (for a horizontally mixed aeration tank system an average velocity of 0.3 m/s must be maintained;
3. Meet maximum oxygen demand and maintain process performance with the largest unit out of service;
4. Provide for varying the amount of oxygen transferred in proportion to the load demand on the plant; and
5. Provide that motors, gear housing, bearings, grease fittings, etc., be easily accessible and protected from inundation and spray as necessary for proper functioning of the unit.

5.8.1.6.3.3.3. Winter Protection

Where extended cold weather conditions occur, the aerator mechanism and associated structure shall be protected from freezing due to splashing. Due to high heat loss, the mechanism as well as subsequent treatment units shall be protected from freezing where extended cold weather conditions occur.

5.8.1.7. Process Definitions

The following are brief descriptions of a number of modifications of the activated sludge process.

5.8.1.7.1. Conventional Activated Sludge

The plug flow activated sludge process is a biological mechanism capable of removing 85 to 95% BOD from typical municipal wastewater. The flow pattern is plug-flow-type. The process is characterized by 20 to 45% sludge return. This is the original activated sludge process and was later modified to suit various applications, situations and treatment requirements. One characteristic of the plug flow configuration is a very high organic loading on the mixed liquor suspended solids (MLSS) in the initial part of the task. Plug flow configurations are often preferred when high effluent DO's are sought.

5.8.1.7.2. Complete Mix Activated Sludge

In a complete mix activated sludge process, the characteristics of the mixed liquor are similar throughout the aeration tank. That is, the influent waste is rapidly distributed throughout the tank and the operating characteristics measured in terms of solids, oxygen uptake rate (OUR), MLSS, and soluble BOD_5 concentration are identical throughout the tank. Because the entire tank contents are the same quality as the tank effluent, there is a very low level of food available at any time to a large mass of microorganisms. This is the major reason why the complete mix modification can handle surges in the organic loading without producing a change in effluent quality.

5.8.1.7.3. Step Aeration

Step feed is a modification of the plug flow configuration in which the secondary influent is fed at two or more points along the length of the aeration tank. With this arrangement, oxygen uptake requirements are relatively even, resulting in better utilization of the oxygen supplied. Step feed configurations generally use diffused aeration equipment. Secondary influent flow is usually added in the first 50 to 75% of the aeration tank's length.

5.8.1.7.4. Contact Stabilization

Contact stabilization activated sludge is both a process and a specific tankage configuration. Contact stabilization encompasses a short-term contact tank, secondary clarifier, and a sludge stabilization tank with about six times the detention time used in the contact tank. This unit operation was developed to take advantage of the fact that BOD removal occurs in two stages. The first is the absorptive phase and the second is the stabilization of the absorbed organics. Contact stabilization is best for smaller flows in which the mean cell residence time (MCRT) desired is quite long. Therefore, aerating return sludge can reduce tank requirements by as much as 30 to 40% versus that required in an extended aeration system.

5.8.1.7.5. Extended Aeration

The extended aeration process used the same flow scheme as the complete mix or plug flow processes but retains the wastewater in the aeration tank for long periods of time. This process operates at a high MCRT (low F/M) resulting in a condition where there is not enough food in the system to support all the microorganisms present. The microorganisms therefore compete very actively for the remaining food and even use their own cell structure for food. This highly competitive situation results in a highly treated effluent with low sludge production. However, extended aeration plant effluents generally have significant concentrations of "pin floc" resulting in BOD_5 and SS removals of about 85%. Many extended aeration systems do not have primary clarifiers. Also, many are package plants used by small communities.

The main disadvantages of this system are the large oxygen requirements per unit of waste entering the plant and the large tank volume needed to hold the wastes for the extended period.

5.8.1.7.6. Oxidation Ditch

The oxidation ditch is a variation of the extended aeration process. The wastewater is pumped around a circular or oval pathway by a mechanical aerator/pumping device at one or more points along the flow pathway. In the aeration tank, the mixed liquor velocity is maintained between 0.2 to 0.37 m/s in the channel to prevent solids from settling.

Oxidation ditches use mechanical brush disk aerators, surface aerators, and jet aerator devices to aerate and pump the liquid flow.

5.8.1.7.7. High Rate Aeration

This is a type of short-term aeration process in which relatively high concentrations of MLSS are maintained, by utilizing high sludge re-circulation rates (100 to 500%), and low hydraulic retention times. Depending on the excess sludge wasting procedure, 60 to 90% BOD removal is achieved for normal domestic wastes. This process is usually (but not necessarily) accomplished in "combined-tank" units.

5.8.1.7.8. High Purity Oxygen

The most common high purity oxygen activated sludge process uses a covered and staged aeration tank configuration. The wastewater, return sludge, and oxygen feed gas enter the first stage of this system and flow concurrently through the tank. The tanks in this system are covered to retain the oxygen gas and permit a high degree of oxygen use. A prime advantage of the staged reactor configuration of the oxygenation system is the system's ability to match approximately the biological uptake rate with the available oxygen gas purity.

5.8.1.8. Return Sludge Equipment

5.8.1.8.1. Return Sludge Rate

The minimum permissible return sludge rate of withdrawal from the final settling tank is a function of the concentration of suspended solids in the mixed liquor entering it, the sludge volume index of these solids, and the length of time these solids are retained in the settling tank. Since undue retention of solids in the final settling tanks may be deleterious to both the aeration and sedimentation phases of the activated sludge process, the rate of sludge return expressed as a percentage of the average design flow of wastewater should generally be variable between the limits set forth in Table 5.18.

The rate of sludge return shall be varied by means of variable speed motors, drives, or timers (small plants) to pump sludge at the above rates.

Table 5.18
Percentage of Average Design Flow

Type of Process	Minimum	Normal	Maximum
Plug Flow	25	30	100
Complete Mix	25	30	100
Carbonaceous Stage of Separate Stage	25		75
Stage Nitrification	25	50	75
Step Aeration	50	100	150
Contact Stabilization	50	100	150
Extended Aeration	50	100	150
Oxidation Ditch	50	50	200
High Rate Nitrification Stage of Separate Stage Nitrification	50		200

5.8.1.8.2. Return Sludge Pumps
If motor driven return sludge pumps are used, the maximum return sludge capacity should be obtained with the largest pump out of service. A positive pressure should be provided on pump suctions. Pumps should have at least 100 mm suction and discharge openings.

If air lifts are used for returning sludge from each settling tank hopper, no standby unit will be required provided the design of the air lifts are such to facilitate their rapid and easy cleaning and provided other suitable standby measures are provided. Air lifts should be at least 100 mm in diameter.

5.8.1.8.3. Return Sludge Piping
Discharge piping should be at least 100 mm in diameter and should be designed to maintain a velocity of not less than 0.6 m/s when return sludge facilities are operating at normal return sludge rates. Suitable devices for observing, sampling, and controlling return activated sludge flow from each settling tank hopper shall be provided, as outlined in Section 5.6.1.3.2.5.

5.8.1.8.4. Waste Sludge Facilities
Waste sludge control facilities should have a maximum capacity of not less than 25% of the average rate of wastewater flow and function satisfactorily at rates of 0.5% of average wastewater flow or a minimum of 0.63 L/s, whichever is larger. Means for observing, measuring, sampling and controlling waste activated sludge flow shall be provided. Waste sludge may be discharged to the concentration or thickening tank, primary settling tank, sludge digestion tank, vacuum filters, or any practical combination of these units.

5.8.1.8.5. Froth Control Units

It is essential to include some means of controlling froth formation in all aeration tanks. A series of spray nozzles may be fixed on top of the aeration tank. Screened effluent or tap water may be sprayed through these nozzles (either continuously or on a time clock on-off cycle) to physically break up the foam. Provision may be made to use antifoaming chemical agents into the inlet of the aeration tank or preferably into the spray water.

5.8.1.9. Measuring Devices

Devices should be installed in all plants for indicating flow rates of raw wastewater or primary effluent, return sludge, and air to each tank unit. For plants designed for wastewater flows of 450 m^3/d or more, these devices should totalize and record, as well as indicate flows. Where the design provides for all return sludge to be mixed with the raw wastewater (or primary effluent) at one location, then the mixed liquor flow rate to each aeration unit should be measured.

5.8.2. Rotating Biological Contactors

5.8.2.1. Applicability

The Rotating Biological Contactor (RBC) process may be used where wastewater is amenable to biological treatment. The process may be used to accomplish carbonaceous and/or nitrogenous oxygen demand reductions.

Considerations for the rotating biological contactor (RBC) process should include:

1. Raw sewage amenability to biological treatment;
2. Pretreatment effectiveness including scum and grease removal;
3. Expected organic loadings, including variations;
4. Expected hydraulic loadings, including variations;
5. Treatment requirements, including necessary reduction of carbonaceous and/or nitrogenous oxygen demand;
6. Sewage characteristics, including pH, temperature, toxicity, nutrients;
7. Maximum organic loading rate of active disc surface area; and
8. Minimum detention time at maximum design flow.

5.8.2.2. Winter Protection

Wastewater temperature affects rotating contactor performance. Year-round operation in colder climates requires that rotating contactors be covered to protect the biological growth from cold temperatures and the excessive loss of heat from the wastewater with the resulting loss of performance.

Enclosures shall be constructed of a suitable corrosion resistant material. Windows or simple louvred mechanisms, which can be opened in the summer and closed in the winter, shall be installed to provide adequate ventilation. To minimize condensation, the enclosure should be adequately insulated and/or heated. Mechanical ventilation should be supplied when the RBC's are contained within a building provided with interior access for personnel.

5.8.2.3. Flow Equalization
For economy of scale, the peaking factor of maximum flow to average daily flow should not exceed 3. Flow equalization should be considered in any instance where the peaking factor exceeds 2.5.

5.8.2.4. Operating Temperature
The temperature of wastewater entering any RBC should not drop below 13°C unless there is sufficient flexibility to decrease the hydraulic loading rate or the units have been increased in size to accommodate the lower temperature. Otherwise, insulation or additional heating must be provided to the plant.

5.8.2.5. Design Flexibility
Adequate flexibility in process operation should be provided by considering one or more of the following:
1. Variable rotational speeds in first and second stages;
2. Multiple treatment trains;
3. Removable baffles between all stages;
4. Positive influent flow control to each unit or flow train;
5. Positively controlled alternate flow distribution systems;
6. Positive airflow metering and control to each shaft when supplemental operation or air drive units are used; and
7. Recirculation of secondary clarifier effluent.

5.8.2.6. Hydrogen Sulphide
When higher than normal influent or sidestream hydrogen sulphide concentrations are anticipated, appropriate modifications in the design should be made.

5.8.2.7. Pretreatment
RBC's must be preceded by effective settling tanks equipped with scum and grease collecting devices unless substantial justification is submitted for other pretreatment devices, which provide for effective removal of grit, debris and excessive oil or grease prior to the RBC units. Bar screening or comminution are not suitable as the sole means of pretreatment.

5.8.2.8. Unit Sizing
The Designer of an RBC system shall conform to the following design criteria, unless it can be shown by thorough documentation that other values or procedures are appropriate. This documentation may include detailed design calculations, pilot test results, and/or manufacturer's empirical design procedures. It should be noted that use of manufacturer's design procedures should be tempered with the realization that they are not always accurate and in some cases can substantially overestimate attainable removals.

Unit sizing shall be based on experience at similar full-scale installations or thoroughly documented pilot testing with the particular wastewater. In determining design loading rates, expressed in units of volume per day per unit area of media covered by biological growth, the following parameters must be considered:

1. Design flow rate and influent waste strength;
2. Percentage of BOD to be removed;
3. Media arrangement, including number of stages and unit areas in each stage;
4. Rotational velocity of the media;
5. Retention time within the tank containing the media;
6. Wastewater temperature; and
7. Percentage of influent BOD, which is soluble.

In addition to the above parameters, loading rates for nitrification will depend upon influent total kjeldahl nitrogen (TKN), pH, and the allowable effluent ammonia nitrogen concentration.

5.8.2.9. Hydraulic Loading

Hydraulic loading to the RBC's should range from 75 to 155 L/m^2d of media surface area without nitrification, and 30 to 80 L/s with nitrification.

5.8.2.10. Organic Loading

The RBC process is approximately first order with respect to BOD removal; i.e. for a given hydraulic loading (or retention time) a specific percent BOD reduction will occur, regardless of the influent BOD concentration. However, BOD concentration does have a moderate effect on the degree of treatment, and thus the possibility of organic overloading in the first stage. With this in mind, organic loading to the first stage of an RBC train should not exceed 0.03 to 0.04 kg BOD/m^2d or 0.012 to 0.02 kg BOD $soluble/m^2d$.

Loadings in the higher end of these ranges will increase the likelihood of developing problems such as heavier than normal biofilm thickness, depletion of dissolved oxygen, nuisance organisms, and deterioration of overall process performance. The structural capacity of the shaft; provisions for stripping biomass; consistently low influent levels of sulphur compounds to the RBC units; the media surface area required in the remaining stages; and the ability to vary the operational mode of the facility may justify choosing a loading in the high end of the range, but the operator must carefully monitor process operations.

5.8.2.11. Tank Volume

For purposes of plant design, the optimum tank volume is measured as wastewater volume held within a tank containing a shaft of media per unit of growth-covered surface on the shaft, or L/m^2. The optimum tank volume determined when treating domestic wastewater up to 300 mg/L BOD is 0.042 L/m^2, which takes into account wastewater, displaced by the media and attached biomass. The use of tank volumes in excess of 0.042 L/m^2 does not yield corresponding increases in treatment capacity when treating wastewater in this concentration range.

5.8.2.12. Detention Time

Based on a tank volume of 0.042 L/m^2, the detention time in each RBC stage should range between 40 to 120 minutes without nitrification, and 90 to 250 minutes with nitrification.

5.8.2.13. Media Submergence and Clearance

RBC's should operate at a submergence of approximately 40% based on total media surface area. To avoid possible shaft overstressing and inadequate media wetting, the liquid operating level should never drop below 35% submergence. Media submergence of up to 95% may be allowed if supplemental air is provided. A clearance of 10 to 23 cm between the tank floor and the bottom of the rotating media should be provided so as to maintain sufficient bottom velocities to prevent solids deposition in the tank.

5.8.2.14. Design Considerations

5.8.2.14.1. Unit Staging

The arrangement of media in a series of stages has been shown to significantly increase treatment efficiency. It is therefore recommended that an RBC plant be constructed in at least four stages for each flow path (or four zones of media area). Four stages may be provided on a single unit by providing baffles within the tank. For small installations where the total area requirements dictate two units per flow path, two units may be placed in series with a single baffle in each tank, thus providing the minimum of four stages. For larger installations requiring four or more units per flow path, the units may be placed in a series within the flow path, with each unit itself serving as a single stage. Generally, though, plants requiring more than four stages should be constructed in a series of parallel floor trains, each comprised of four separate stages. Wastewater flow to RBC units may be either perpendicular or parallel to the media shafts.

5.8.2.14.2. Tankage

RBC units may be placed in either steel or concrete tankage with baffles, when required, and constructed of a variety of materials. The design of the tankage must include:

1. Adequate structural support for the RBC and drive unit;
2. Elimination of the "dead" areas;
3. Satisfactory hydraulic transfer capacity between stages of units; and
4. Considerations for operator safety.

The structure should be designed to withstand the increased loads which could result if the tank were to be suddenly dewatered with a full biological growth on the RBC units. The sudden loss of buoyancy resulting from unexpected tank dewatering could increase the bearing support loadings by as much as 40%.

Provisions for operator protection can be included in the tankage design by setting the top of the RBC tankage about one foot above the surrounding floor and walkways, with handrails placed along the top of the tankage, to provide an effective barrier between the operator and exposed moving equipment. The high tank walls will also prevent loss or damage by any material accidentally dropped in the vicinity of the units and entering the tankage.

5.8.2.14.3. High Density Media

Except under special circumstances, high-density media should not be used in the first stage. Its use in subsequent stages should be based on appropriate loading criteria, structural limitations of the shaft and media, and media configuration.

5.8.2.14.4. Shaft Rotational Velocity

The peripheral velocity of a rotating shaft should be approximately 18 m/min for mechanically driven shaft, and between 9 and 18 m/min for an air driven shaft. Provision should also be made for rotational speed control and reversal.

5.8.2.14.5. Biomass Removal

A means for removing excess biofilm growth should be provided, such as air or water stripping, chemical additives, rotational speed control/reversal, etc.

5.8.2.14.6. Dissolved Oxygen Monitoring

First-stage dissolved oxygen (DO) monitoring should be provided. The RBC should be able to maintain a positive DO level in all stages.

5.8.2.14.7. Supplemental Air

Periodic high organic loadings may require supplemental aeration in the first stage to promote sloughing of biomass.

5.8.2.14.8. Side Stream Inflows

The type and nature of side stream discharges to an RBC must be evaluated, and the resulting loads must be added to the total facility influent loads. Anaerobic digesters increase ammonia nitrogen loadings, and sludge conditioning processes such as heat treatment contribute increased organic and ammonia nitrogen loadings. Whenever septic tank discharges comprise part of the influent wastewater or any unit processes are employed that may produce sulphide ahead of the RBC units, the additional oxygen demand associated with sulphide must be considered in system design.

5.8.2.14.9. Re-circulation

Consideration should be given to providing re-circulation of RBC effluent flow. This may be necessary during initial start-up and when the inflow rate is reduced to extremes.

For small installations, such as those serving an industrial park or school, the inflow over weekends or at holiday periods may drop to zero. During such periods, the lack of incoming organic load will cause the media biogrowth to enter the endogenous respiration phase where portions of the biogrowth become the food source or substrate for other portions of the biogrowth. If this condition lasts long enough, all of the biogrowth will eventually be destroyed. When this condition is allowed to exist, the RBC process does not have adequate biogrowth to provide the desired treatment when the inflow restarts.

If flow can be recycled through the sludge holding/treatment units and then to the RBC process, an organic load from the sludge units can be imposed on the RBC process. This imposed load will help to maintain the biogrowth and, as a secondary benefit, help stabilize and reduce the sludge.

When any new facility is first started, the biogrowth is slow to establish. If it is desired to build up the biogrowth before directing all of the inflow to the RBC process (as when the RBC is replacing an older existing process) some inflow may be directed to the RBC process and recycled.

In the first few days, minimal biogrowth will develop with only minimal removal of the organic load. By recycling, the unused organic load again becomes available to the biogrowth. As the biogrowth develops, the recycle rate should be reduced, with new inflow added to increase the organic load. As the biogrowth develops further, the recycle is eventually reduced to zero with all of the inflow being the normal RBC influent.

5.8.2.14.10. Load Cells

Load cells, especially in the first stage(s), can provide useful operating and shaft load data. Where parallel trains are in operation, they can pinpoint overloaded or under-loaded trains. Stop motion detectors, rpm indicators and clamp-on ammeters are also potentially useful monitoring instruments.

Therefore, load cells shall be provided for all first and second stage shafts. Load cells for all other shafts in an installation are desirable.

5.8.2.14.11. Shaft Access

In all RBC designs, access to individual shafts for repair or possible removal must be considered. Bearings should also be accessible for easy removal and replacement if necessary. Where all units in a large installation are physically located very close together, it may be necessary to utilize large off-the-road cranes for shaft removal. Crane reach, crane size, and the impact of being able to drain RBC tankage and dry a unit prior to shaft removal should all be considered when designing the RBC layout.

5.8.2.14.12. Structural Design

The designer should require the manufacturer to provide adequate assurance that the shaft and media support structures are protected from structural failure for the design life of the facility. Structural designs should be based on appropriate American Welding Society (AWS) stress category curves modified as necessary to account for the expected corrosive environment. All fabrication during construction should conform to AWS welding and quality control standards.

5.8.2.14.13. Energy Requirements

Energy estimates used for planning and design should be based on expected operating conditions such as temperature, biofilm thickness, rotational speed, type of unit (either mechanical or air driven), and media surface area instead of normalized energy data sometimes supplied by equipment manufacturers. Care should be taken to assure that manufacturer’s data are current and reflect actual field-validated energy usage.

Only high efficiency motors and drive equipment should be specified. The designer should also carefully consider providing power factor correction for all RBC units.

5.8.2.14.14. Nitrification Consideration

Effluent concentrations of ammonia nitrogen from the RBC process designed for nitrification are affected by diurnal load variations. Therefore, it may be necessary to increase the design surface area proportional to the ammonia nitrogen diurnal peaking rates to meet effluent limitations. An alternative is to provide flow equalization sufficient to insure process performance within the required effluent limitations.

5.8.3. Sequencing Batch Reactor (SBR)

The Sequencing Batch Reactor (SBR) is a fill-and-draw activated sludge treatment system. All SBR systems utilize five steps that occur sequentially within the same tank as follows:

1. Fill;
2. React (aeration);
3. Settle (clarification);
4. Decant; and
5. Idle.

Process modifications can be made by varying the times associated with each step, in order to achieve specific treatment objectives. When designing or evaluating SBR systems care must be taken with the processes that are unique to the SBR. These include:

1. Fill Method;
2. Hydraulic Control Systems;
3. Aeration Control Systems;
4. Method of Decant;
5. Sizing of Disinfection Equipment for Decant Flows; and
6. Sludge Wasting Methods.

One of the main strengths of the SBR process is the process flexibility that can be achieved. Therefore, the above processes can be performed using a variety of methods. Designers of SBR systems must be prepared to supply sufficient detailed information at the request of the DOEC.

5.8.3.1. Process Configurations

One classification of SBR systems distinguishes those that operate with continuous feed and intermittent discharge (CFID) from those that operate with intermittent feed and intermittent discharge (IFID).

5.8.3.2. Continuous Influent Systems

Continuous feed-intermittent discharge reactors receive influent wastewater during all phases of the treatment cycle. When there is more than one reactor, as is typically the case for municipal systems, the influent flow is split equally to the various reactors on a continuous basis. For two-reactor systems, it is normal to have the reactor cycle operations displaced so that one SBR is aerating while the second SBR is in the settling and decant phases. This makes it possible to aerate both reactors with one blower continuously in operation and also spreads the decant periods so that there is no overlap. The dry weather flow cycle time for most CFID systems is generally 3 to 4 hours. Each cycle typically devotes 50% of the cycle time to aeration, 25% to settling, and 25% to decant. Storm water flows are accommodated by reducing cycle time. Under extreme flow condition, the reactor may operate as a primary clarifier (no aeration phase) with the decanters set at top water level (TWL).

With a CFID system, TWL occurs at the start of the decant phase. Because CFID systems generally operate on the basis of preset time cycles, TWL varies for each cycle as a function of the influent flow for that particular cycle. The actual effluent flow rate during the discharge event depends on the number of reactors and the percentage of each cycle devoted to decant.

A key design consideration with CFID systems is to minimize short-circuiting between influent and effluent. Influent and effluent discharges are typically located at opposite ends of rectangular reactors, with length-to-width ratios of 2:1 to 4:1 being common. Installation of a pre-reaction chamber separated by a baffle wall from the main reaction chamber is also a standard feature of some systems.

5.8.3.3. Intermittent Influent Systems

IFID types of systems are sometimes referred to as the conventional, or "true," SBR systems. The one common characteristic of all IFID systems is that the influent flow to the reactor is discontinued for some portion of each cycle.

In IFID systems each reactor operates with five discrete phases during a cycle. During the period of reactor fill, any combination of aeration, mixing, and quiescent filling may be practiced. Mixing independent of aeration can be accomplished by using jet aeration pumps or separate mixers. Systems should distribute the influent over a portion of the reactor bottom so that it will contact settled solids during unaerated and unmixed fill. The end of the fill cycle is controlled either by time (that is, fill for a preset length of time) or by volume (that is, fill until the water level rises a fixed amount). Flow information from the WWTP influent flow measurement or from the rise rate in the reactor determined by a series of floats may be used to control the time allocated to aeration, mixing, or filling in accordance with previously programmed instructions.

At the end of the fill cycle, all influent flow to the first reactor is stopped, and flow is diverted to the second reactor. Continuous aeration occurs during the react phase for a predetermined time period (typically 1 to 3 hours). Again, the time devoted to reaction in any given cycle may automatically be changed as a function of influent flow rate. At the completion of the reaction phase, aeration and any supplemental mixing are stopped, and the mixed liquor is allowed to settle under quiescent conditions (typically 30 to 60 minutes). Next, clarified effluent is decanted until the bottom water level (BWL) is reached. The idle period represents the time period between the end of decant and the time when influent flow is again redirected to a given reactor. During high-flow periods, the time in idle will typically be minimal.

The actual flow rate during discharge has the potential to be several times higher than the influent flow rate. Discharge flow rates are critical design parameters for the downstream hydraulic capacity of sewers (in the case of industrial treatment facilities) or processes such as disinfection or filtration.

Another variation of the IFID approach dispenses with a dedicated reaction phase and initiates the settling cycle at the end of aerated fill. Yet another IFID approach allows influent to enter the reactor at all times except for the decant phase so that normal system operation consists of the following phases:

1. Fill-aeration;
2. Fill-settling;
3. No fill-decant; and
4. Fill-idle.

These systems also include an initial selector compartment that operates either at constant or variable volume and serves as a flow splitter in multiple-basin systems. Biomass is directed from the main aeration zone to the selector.

Sequencing batch reactor systems can also be designed for nitrification-denitrification and enhanced biological phosphorus removal. In these cases, the cycle times devoted to such processes as anaerobic fill, anoxic fill, mixed/unmixed fill, aerobic fill, and dedicated reaction depend on the treatment objectives. Mineral addition may also be practiced to achieve effluent objectives more stringent than typical secondary effluent requirements. Systems can also be configured to switch from IFID operation to CFID operation when necessary to accommodate storm water flows or to allow a basin to be removed from service while still treating the entire WWTP flow in a remaining basin. The one common factor behind all SBRs is that aeration, settling, and decant occur within the same reactor.

5.8.3.4. Sequencing Batch Reactor Equipment

5.8.3.4.1 Process Control

The programmable logic controller (PLC) is the optimum tool for SBR control and all present-day vendors use this approach. Sequencing batch reactor manufacturers supply both the PLC and required software. Typically, programs are developed and modified by the SBR vendor using a desktop computer and software supplied by the PLC vendor. Vendor-developed programs are proprietary and may not be modified by the design engineer or the WWTP operator. Depending on the proprietary software design and type of system, the operator may independently select such variables as solids waste rates; storm cycle times; and aeration, mixing, and idle times. In addition, the design engineer may develop additional software to interface to PLC to a desktop computer for graphic presentation of process operation to the operator and generation of archive date and compliance reports.

Programmable logic controller hardware is of modular construction. Troubleshooting procedures are well defined, and replacement of a faulty module is not difficult. An internal battery protects the software in the event of power failure. The software is backed up by a memory chip (EPROM) and can be easily reloaded if the battery fails. The PLC expertise required of the owner is limited to maintenance and repair functions that are well within the capability of a competent electrician.

5.8.3.4.2. Reactors

Reactor shapes include rectangular, oval, circular, sloped sidewall, and other unique approaches. Design TWLs and BWLs often allow decanting from 20 to 30% of the reactor contents per cycle.

5.8.3.4.3. Decanters

Some decanters are mechanically actuated surface skimmers that typically rest above the TWL. The decanter is attached to the discharge pipe by smaller pipes that both support and drain the decanter. The discharge pipe is coupled at each end through seals that allow it to rotate. A screw-type jack attached to a worm tear, sprocket, and chain to an electric motor rotates the decanter from above the TWL to BWL. The speed of rotation is adjustable.

Other decanters are floated on the reactor surface. These decanters may approximate a large-diameter plug valve, whereby the top portion acts as the valve seat (and provides flotation). The bottom is the plug that is connected to a hydraulic operator that moves it away from the seat to allow discharge, or back to the seat to stop discharge. Other floating decanters consist of a length of pipe suspended on floats, with the pipe having a number of orifices bored in the bottom. The number of orifices (and length of pipe) is flow dependent. Each orifice is blocked by a flapper or plugs to prevent solids entry during aeration. There are also decanter configurations that float an effluent discharge pump.

Other decanters are typically fixed-position siphons located on the reactor wall. The bottom of the decanter (collection end of the siphon) is positioned at the BWL. Flow into the decanter is under a front lip (scum baffle), over an internal dam, and out through a valve. When the water level in the reactor falls below the front lip, air enters the decanter, breaking the siphon and stopping flow.

The trapped air prevents mixed liquor from entering during the reaction and settling modes. At the end of settling, the trapped air is released through a solenoid valve and the siphon is started.

5.8.3.4.4. Solids Wasting

The wasting of both aerated mixed liquor suspended solids (MLSS) and settled MLSS is practiced. The wasting systems frequently consist of a submersible pump with a single point for withdrawal. Gravity flow waste systems are also used. Another approach uses influent distribution piping for multiple-point with withdrawal of the settled solids.

5.8.3.4.5. Aeration/Mixing Systems

A variety of aeration and mixing systems are in use with SBRs. These include jet aeration, fine- and course-bubble aeration, and turbine mechanical aeration. Some systems use a floating mixer to provide mixing independent of aeration. Other diffused aeration facilities do not have any mixing capability independent of aeration. Independent mixing is readily obtained with a jet aeration system.

5.9. Wastewater Treatment Ponds (Lagoons)

This section deals with generally used variations of treatment lagoons to achieve secondary treatment including controlled-discharge lagoon systems, flow-through lagoon systems and aerated lagoon systems. Lagoons utilized for equalization, percolation, evaporation and sludge storage will not be discussed in this section.

5.9.1. Supplement to the Pre-design Report

The Pre-design report shall contain pertinent information on location, geology, soil conditions, area for expansion and any other factors that will affect the feasibility and acceptability of the proposed project. The following sub-sections detail the information that must be submitted in addition to that required in Section 2.

5.9.1.1. Supplementary Field Survey Data

5.9.1.1.1. Location of Nearby Facilities

The location and direction of all residences, commercial developments, parks, recreational areas, and water supplies within l.6 km of the proposed lagoon shall be included in the Pre-design report.

5.9.1.1.2. Land Use Zoning

Land use zoning adjacent to the proposed lagoon site shall be included.

5.9.1.1.3. Site Description

A description, including maps showing elevations and contours of the site and adjacent area shall be provided. Due consideration shall be given to additional treatment units and/or increased waste loadings in determining land requirements.

5.9.1.1.4. Location of Field Tile

The location, depth, and discharge point of any field tile in the immediate area of the proposed site shall be identified.

5.9.1.1.5. Soil Borings

Data from soil borings conducted by an independent soil-testing laboratory to determine subsurface soil characteristics and groundwater characteristics (including elevation and flow) of the proposed site and their effect on the construction and operation of a lagoon shall also be provided. At least one boring shall be a minimum of 7.6 m in depth, or into bedrock, whichever is shallower. If bedrock is encountered, rock type, structure and corresponding geological formation data should be provided. The boring shall be filled and sealed. The permeability characteristics of the lagoon bottom and lagoon seal materials shall also be studied (see Section 5.9.6.2).

5.9.1.1.6. Sulphate Content of Water Supply

Sulphate content of the basic water supply shall be determined.

5.9.1.1.7. Percolation Rates

Data demonstrating anticipated percolation rates at the elevation of the proposed lagoon bottom shall be included.

5.9.1.1.8. Well Survey

A pre-construction survey of all nearby wells (water level and water quality) is mandatory.

5.9.2. Location

5.9.2.1. Distance From Habitation

A lagoon site should be located as far as practicable, with a minimum of 150 m from isolated habitation and 300 m from built up areas or areas, which may be built up within a reasonable future period. Consideration should be given to site specifics such as topography, prevailing winds, forests, etc.

A minimum distance of 100 m from public roads and highways is also recommended. Aerated stabilization basins separation distances shall be considered the same as mechanical plants.

5.9.2.2. Prevailing Winds

If practicable, lagoons should be located so that local prevailing winds will be in the direction of uninhabited areas.

5.9.2.3. Surface Runoff

Location of lagoons in watersheds receiving significant amounts of stormwater runoff is discouraged. Adequate provision must be made to divert stormwater runoff around the lagoons and protect pond embankments from erosion.

5.9.2.4. Groundwater Pollution

Existing wells, which serve as drinking water sources should be protected from health hazards. Possible travel of pollutants through porous soils and fissured rocks should be objectively evaluated to safeguard the wells. A lagoon shall be located as far as practicable, with a minimum of 300 m from any well used as a drinking water source.

A minimum separation of 1.2 m between the bottom of the lagoon and the maximum groundwater elevation should be maintained, however, less separation may be acceptable when supported by appropriate hydrogeological and engineering designs/investigations upon acceptance of the DOEC.

A minimum of 1.5 m between the bottom of the lagoon and bedrock is recommended, however, less separation may be acceptable when supported by appropriate hydrogeological and engineering designs/investigations upon acceptance of the DOEC.

5.9.2.5. Protection of Surface Water Supplies
Lagoons shall be located downhill, downstream and remote from all sources of surface water supplies (lakes and rivers). The minimum distances outlined in Table 5.19 shall be employed as the criteria:

Table 5.19
Minimum Distance Criteria

Minimum Distance from a Lake or River to the Centre of a Dyke of a Proposed Lagoon	Remarks
120 m	Lined lagoon, pervious soil
75 m	Lined lagoon, impervious soil

5.9.2.6. Geology
Lagoons shall not be located in areas, which may be subjected to karstification (i.e. sink holes or underground streams generally occurring in areas underlain by limestone or dolomite).

A minimum separation of 3.0 m between the lagoon bottom and any bedrock formation is recommended.

5.9.2.7. Floodplains
A lagoon shall not be located within the 100-year floodplain.

5.9.3. Definitions
Aerobic Lagoon - Aerobic lagoons are shallow basins used for wastewater treatment. The organic contaminants in the wastewater are degraded by aerobic and facultative bacteria. The lagoons characteristically receive a light organic loading. They are used primarily to achieve additional organic removal following conventional wastewater treatment. Dissolved oxygen is furnished by oxygen transfer between the air and water surface, and by photosynthetic algae. The amount of oxygen supplied by natural surface re-aeration depends largely on wind-induced turbulence.

Facultative Lagoons - The facultative lagoon is divided into an aerobic layer at the top and an anaerobic layer on the bottom. The aerobic layer is generated by algae, which produce oxygen by photosynthesis. Settleable solids are permitted to accumulate on the pond bottom, and are broken down anaerobically. Waste stabilization is accomplished by a combination of anaerobic, aerobic, and a preponderance of facultative organics interacting with the wastewater.

Aerated Lagoons - An aerated lagoon may be aerated aerobic (completely mixed) or aerated facultative. It does not depend on algae and sunlight to furnish DO or bacterial respiration, but instead uses diffusers or other mechanical aeration devices to transfer the major portion of oxygen and to create some degree of mixing. Because of the mixing, removal of suspended solids in the lagoon effluent is an important consideration. Aerated lagoons can be described as heavily loaded oxidation basins, or very lightly loaded activated sludge systems. The microorganisms responsible for the organic breakdown tend to be similar to those found in activated sludge systems.

Aerated lagoons may be used in series with aerobic lagoons. In such cases, the primary purpose of the lagoon without aeration is for solids removal.

5.9.4. Application, Advantages and Disadvantages of Different Lagoon Types

Table 5.20 details the appropriate application of various lagoon types, as well as summarizes the advantages and disadvantages related to each type.

Table 5.20
Application, Advantages and Disadvantages of the Different Lagoon Types

Parameter	Unaerated Aerobic	Facultative	Aerated	
			Aerobic	**Facultative**
Application	Nutrient Removal; Treatment of Soluble Organic Wastes; Secondary Effluents	Treatment of raw domestic and industrial wastes	Treatment of raw domestic and industrial wastes	Treatment of raw domestic and industrial wastes
Advantages	Low operating and maintenance costs	Low operating and maintenance costs	Small volume and area; resistance to upsets	Small volume and area; resistance to upsets
Disadvantages	Large volume and area; possible odours	Large volume and area; possible odours	Significant maintenance and operating costs; high solids in effluent; foaming	Maintenance and operation costs; foaming

5.9.5. Basis of Design

5.9.5.1. Lagoons

5.9.5.1.1. Holding Capacity Requirements

Before the design of a lagoon system can be initiated, the designer shall determine the following:

1. Whether the lagoon can be continuously discharged or must operate on a fill-and-draw basis;

2. The period of the year if any, when discharge will not be permitted;

3. What discharge rates will be permitted with fill-and-draw lagoons and what, if any, provision must be made for controlling effluent discharge rates in proportion to receiving stream flow rates; and

4. What the minimum time for discharge of lagoon cell contents should be for fill-and-draw systems.

The holding capacity of lagoons shall be based upon average daily sewage flow rates, making a special allowance for net precipitation entering the cells.

5.9.5.1.2. Area and Loadings

One hectare of water surface should be provided for each 250-design population or population equivalent. In terms of BOD, a loading of 22 kg BOD_5/ha·day should not be exceeded. Higher or lower design loadings will be judged after review of material contained in the Pre-Design report and after a field investigation of the proposed site by the DOEC. Due consideration shall be given to possible future municipal expansion and/or additional sources of wastes when the original land acquisition is made. Suitable land should be available at the site for increasing the size of the original construction.

Where substantial ice cover may be expected for an extended period, it may be desirable to operate the facility to completely retain wintertime flows. Design variables such as lagoon depth, multiple units, detention time and additional treatment units must be considered with respect to applicable standards for BOD_5, total suspended solids (TSS), fecal coliforms, dissolved oxygen (DO) and pH.

5.9.5.1.3. Flow Distribution

The main inlet sewer or forcemain should terminate at a chamber, which permits hydraulic and organic load splitting between the lagoon cells. The ability to introduce raw sewage to all cells is desirable, but as a minimum, there must be a capability to divide raw sewage flows between enough cells to reduce the BOD_5 loading to 22 kg BOD_5/ha·day, or less. The inlet chamber should be provided with a lockable aluminum cover plate or grating, divided into small enough sections to permit easy handling.

5.9.5.1.4. Typical Performance Potentials

Atlantic region environmental conditions are expected to facilitate the following performance of lagoons treating typical domestic wastes:

Winter efficiency = 70% BOD removal
Summer efficiency = 80% BOD removal
Organic Load = 22 kg BOD_5/ha·day
Liquid depth = 1.5 to 1.8 m
Suspended Solids removal = 80% but may decrease with increasing algal concentrations

5.9.5.1.5. Controlled - Discharge Lagoons

For controlled-discharge systems, the area specified as the primary ponds should be equally divided into two cells. The third or secondary cell volume should, as a minimum, be equal to the volume of each of the primary cells. In addition, the design should permit for adequate elevation difference between primary and secondary ponds to permit gravity filling of the secondary from the primary. Where this is not feasible, pumping facilities may be provided.

5.9.5.1.6. Flow-Through Lagoons
At a minimum, primary cells shall provide adequate detention time to maximize BOD removal. Secondary cells should then be provided for additional detention time with depths to 2.0 m to facilitate both solids and coliform reduction.

5.9.5.1.7. Tertiary Lagoon
When lagoons are used to provide additional treatment for effluents from existing or new secondary sewage treatment works, the DOEC will, upon request, establish BOD loadings for the lagoon after due consideration of the efficiencies of the preceding treatment units.

5.9.5.2. Aerated Lagoons
Aerated lagoons can be either aerobic or facultative. An aerated aerobic lagoon contains dissolved oxygen through the whole system with no anaerobic zones. The lagoon shape and the aerating power provide complete mixing. The aerated facultative lagoon provides a partially mixed condition, which will cause an anaerobic zone to develop at the bottom as suspended solids settle due to low velocity in the system.

5.9.5.2.1. Aerated Aerobic Lagoons
In general, an aerated lagoon can be classified as an aerobic lagoon (complete mixed) if the mechanical aeration power level is above six watts per cubic meters of maximum storage. Aerated aerobic lagoons should be designed to maintain complete mixing with bottom velocities of at least 0.15 m/s. It is important that sufficient mixing power be provided.

Quiescent settling areas adjacent to the aerated cell outlets or the addition of suspended solids removal processes such as a clarifier must follow aerated aerobic treatment, to insure compliance with suspended solids discharge requirements. In most cases, a minimum detention time of one day is required to achieve solids separation. Algae growth should be limited by controlling the hydraulic detention time to two days or less. Water depth of not less than 1.0 m shall be maintained to control odours arising from anaerobic decomposition. Adequate provision must be made for sludge storage so that the accumulated solids will not reduce the actual detention time.

5.9.5.2.2. Aerated Facultative Lagoons
Aerated facultative lagoons should be designed to maintain a minimum of 2.0 mg/L of dissolved oxygen (DO) in the upper zone of the liquid. The aeration system must be able to transfer up to 1.0 kg of oxygen per kg of BOD_5 applied uniformly throughout the pond when the water temperature is 20°C. The organic loading rate should be maintained between 0.031 and 0.048 kg/m^3day.

The escape of algae into the effluent should be controlled by providing a quiescent area adjacent to each cell outlet with an overflow rate of 32 m^3/m^2d. If multiple aerated facultative cells are used, all cells following the first one shall have diminished aeration capacity to permit additional settling. Whenever possible, provisions should be provided for re-circulating part (5-10%) of the final aeration cell effluent back into the influent in order to maintain a satisfactory mix of active microorganisms.

5.9.5.2.3. Design Parameters

5.9.5.2.3.1. Detention Time

The mean cell residence time of an aerated lagoon should ensure that the suspended micro-organisms have adequate detention time to transform non-settling and dissolved solids into settleable solids, an adequate factor of safety is provided for periods of high hydraulic loading and that the detention time in the aerated lagoon is controlled by the rate of metabolism during the coldest period of the year.

As a minimum, the detention time should reflect 85% BOD_5 removal from November to April, being based on good and efficient operation of the aeration equipment. For the development of final design parameters, it is recommended that actual experimental data be developed; however, the aerated lagoon system design for minimum detention time may be estimated using the following formula:

$$t = \frac{E}{2.3K1 \times (100 - E)}$$

where: t = detention time (days);

E = percent of BOD5 to be removed in an aerated lagoon; and

K1 = reaction coefficient, aerated lagoon, base 10. For normal domestic sewage, the K1 value may be assumed to be 0.12/d at 20°C and 0.06/d at 1°C.

The reaction rate coefficient for domestic sewage which includes some industrial wastes, other wastes and partially treated sewage must be determined experimentally for various conditions which might be encountered in the aerated lagoons. Conversion of the reaction rate coefficient at other temperatures shall be made based on experimental data. Additional storage volume should be considered for sludge and ice cover.

5.9.5.2.3.2. Oxygen Requirement

Oxygen requirements generally will depend on the BOD loading, the degree of treatment and the concentration of suspended solids to be maintained. Aeration equipment shall be capable of maintaining a minimum dissolved oxygen level of 2.0 mg/L in the lagoons at all times. The oxygen requirements should meet or exceed the peak 24-hour summer loadings. A safety factor of up to 2 should be considered in designing oxygen supply equipment based on average BOD_5 loadings. The amount of oxygen requirement has been found to vary from 0.7 to 1.5 times the amount of BOD_5 removed. Suitable protection from weather shall be provided for electrical control.

5.9.5.3. Industrial Wastes

Consideration shall be given to the type and effects of industrial wastes on the treatment process. In some cases it may be necessary to pretreat industrial or other discharges.

Industrial wastes shall not be discharged to lagoons without assessment of the effects such substances may have upon the treatment process or the discharge requirements of the DOEC.

5.9.5.4. Multiple Units

At a minimum, a lagoon system should consist of 2 cells designed to facilitate both series and parallel operations. The maximum size of a lagoon cell should be 5 ha. A one-cell system may be utilized in very small installations. Larger cells may be permitted for bigger installations.

All systems should be designed with piping flexibility to permit isolation of any cell without affecting the transfer and discharge capabilities of the total system. In addition, the ability to discharge the influent waste load to a minimum of 2 cells and/or all primary cells in the system should be provided.

Requirements for multiple units in an aerated lagoon system shall be similar to those in an activated sludge system, including requirements for back-up aeration equipment.

5.9.5.5. Design Depth

The minimum operating depth should be sufficient to prevent growth of aquatic plants and damage to the dykes, control structures, aeration equipment and other appurtenances. In no case should lagoon depths be less than 0.6 m.

5.9.5.5.1. Controlled-Discharge Lagoons

The maximum water depth shall be 1.8 m in primary cells. Greater depths in subsequent cells are permissible although supplemental aeration or mixing may be necessary.

5.9.5.5.2. Flow-Through Lagoons

Maximum normal liquid depth should be 1.5 m.

5.9.5.5.3. Aerated Lagoon Systems

In general, normal water depths vary from 1.2 to 3.6 m when using surface aerators, however, consideration should be given to depths of up to 5.0 m to minimize surface heat losses.

5.9.5.6. Lagoon Shape

Square cells are preferred to long narrow rectangular cells. Round, square or rectangular lagoons with a length not exceeding three times the width are considered most desirable. The long dimension of any pond should not align with the prevailing wind direction. No islands, peninsulas or coves shall be permitted. Dykes should be rounded at corners to minimize accumulations of floating materials. Common-wall dyke construction, wherever possible, is strongly encouraged.

5.9.5.7. Additional Treatment

Consideration should be given in the design stage to the utilization of additional treatment units as may be necessary to meet applicable discharge standards.

5.9.6. Lagoon Construction Details

5.9.6.1. Embankments and Dykes

5.9.6.1.1. Materials

Embankments and dykes shall be constructed of relatively impervious material and compacted to at least 90% Standard Proctor Density to form a stable structure. Vegetation and other unsuitable materials shall be removed from the areas where the embankment is to be placed.

5.9.6.1.2. Top Width

The minimum dyke width shall be 3.0 m to permit access of maintenance vehicles.

5.9.6.1.3. Maximum Slopes

Inner and outer dyke slopes shall not be steeper than 1 vertical to 3 horizontal (1:3).

5.9.6.1.4. Minimum Slopes

Inner slopes should not be flatter than 1 vertical to 4 horizontal (1:4). Flatter slopes can be specified for larger installations because of wave action but have the disadvantage of added shallow areas being conducive to emergent vegetation. Outer slopes shall be sufficient to prevent surface runoff from entering the ponds.

5.9.6.1.5. Freeboard

Minimum freeboard shall be 1.0 m. For very small systems, 0.6 m may be acceptable.

5.9.6.1.6. Erosion Control

1. **Outer Dykes** - The outer dykes shall have a cover layer of at least 100 mm of fertile topsoil to promote establishment of an adequate vegetative cover wherever riprap is not utilized. Adequate vegetation shall be established on dykes from the outside toe to 0.5 m below the top of the embankment as measured on the slope. Perennial-type, low-growing, spreading grasses that minimize erosion and can be mowed are most satisfactory for seeding on dykes. Additional erosion control may also be necessary on the exterior dyke slope to protect the embankment from erosion due to severe flooding of a watercourse.

2. **Inner Dykes** - Alternate erosion control on the interior dyke slopes has become necessary for ponds because of problems associated with mowing equipment not designed to run on slopes as well as a lack of maintenance by the plant owner. The inner dykes shall have a cover of at least 200 mm of pit run gravel or other material graded in a manner to discourage the establishment of any vegetation. The material should be spread on dykes from the inside toe to the top of the embankment. Clean and sound riprap or an acceptable equal shall be placed

from 0.3 m above the high water mark to 0.6 m below the low water mark (measured on the vertical). Maximum size of rock used should not exceed 150 mm.

3. **Top of Embankment** - The top of the embankment used for access around the perimeter of the dykes shall have a cover layer of at least 300 mm of cover material similar to the one described in Section 5.9.6.1.6 (2).

4. **Additional Erosion Protection** - Riprap or some other acceptable method of erosion control is required as a minimum around all piping entrances and exits. For aerated cells the design should ensure erosion protection on the slopes and bottoms in the areas where turbulence will occur.

5. **Erosion Control During Construction** - Effective site erosion control shall be provided during construction as required by this DOEC.

6. **Seeding** - The dykes shall have a cover layer of at least 100 mm of fertile topsoil to promote establishment of an adequate vegetative cover wherever riprap is not utilized. Prior to pre-filling (in accordance with Section 5.9.6.4), adequate vegetation shall be established on dykes from the outside toe to 0.6 m above the pond bottom on the interior as measured on the slope. Perennial-type, low-growing, spreading grasses that minimize erosion and can be mowed are most satisfactory for seeding on dikes. In general, alfalfa and other long-rooted crops should not be used for seeding since the roots of this type are apt to impair the water holding efficiency of the dikes.

5.9.6.2. Lagoon Bottom and Liners

5.9.6.2.1. Location

A minimum separation of 3.0 m between the cell bottom and bedrock is recommended. Cell bottoms should be located at least 1.2 m above the high groundwater level, in order to prevent inflow and/or liner damage.

5.9.6.2.2. Uniformity

The pond bottom should be as level as possible at all points. Finished elevations should not be more than 75 mm from the average elevation of the bottom.

5.9.6.2.3. Vegetation

The bottom shall be cleared of vegetation and debris. Organic material thus removed shall not be used in the dyke core construction. However, suitable topsoil relatively free of debris may be used as cover material on the outer slopes of the embankment as described in Section 5.9.6.1.6. (2).

5.9.6.2.4. Soil

Soil used in constructing the lagoon bottom (not including liner) and dyke cores shall be relatively incompressible and tight and compacted at or up to 4 % above the optimum water content to at least 90 % Standard Proctor Density. Soft pockets that would prevent sufficient compaction of the liner must be sub-excavated and replaced with suitable, compacted fill.

5.9.6.2.5. Liner

Lagoons shall be sealed such that seepage loss through the seal is as low as practicably possible. Liners consisting of soils or bentonite as well as synthetic liners may be considered, provided the permeability, durability and integrity of the proposed material can be satisfactorily demonstrated for anticipated conditions. Results of a testing program which substantiates the adequacy of the proposed liner must be incorporated into and/or accompany the Pre-Design report. Standard ASTM procedures or acceptable similar methods shall be used for all tests. Where clay liners are used, precautions should be taken to avoid erosion and desiccation cracking prior to placing the system in operation.

5.9.6.2.6. Seepage Control Criterion for Clay Liners

The seepage control criterion for municipal wastewater lagoons and aerated lagoons utilizing clay liners specifies a maximum hydraulic conductivity, K, for the lagoon liner as a function of the liner thickness, L, and water depth, D, by the equation:

$$Maximum\,K\left({}^{m}\!/\!{}_{s}\right)=\frac{4.6\times10^{-8}\left({}^{m}\!/\!{}_{s}\right)\times L(m)}{D(m)+L(m)}$$

For example, a compacted clay liner that is 0.5 m thick must have a hydraulic conductivity of about 1.3 x 10^{-8} m/s (1.3 x 10^{-6} cm/s) or less. The "K" obtained by the above expression corresponds to a percolation rate of pond water of less than 40 cubic meters per day per hectare at a water depth of 1.2 metres.

5.9.6.2.7. Seepage Control Criterion for Synthetic Liners

For synthetic liners, seepage loss through the liner shall not exceed the quantity equivalent through an adequate soil liner. For liner durability the minimum liner thickness for a HDPE liner shall be 1.5 mm (60 mil). The liner shall be underlain by a sand layer with a minimum thickness of 150 mm. Special consideration should be given to problems associated with bedding movement on berm slopes. Provision must be made to check integrity or possible leakage of liner by means of lysimeters, sampling stations or access to sub-drainage collection system.

5.9.6.2.8. Site Drainage

Surface drainage must be routed around and away from cells. Field tiles within the area enclosed by the berms must be located and blocked so as to prevent cell content leakage. Measures must be taken, where necessary, to avoid disruption of field tile and surface drainage of adjacent lands, by constructing drainage works to carry water around the site.

5.9.6.3. Design and Construction Procedures for Clay Liners

5.9.6.3.1. Delineation of Borrow Deposit

The first step in designing a compacted clay liner is delineating a relatively uniform deposit of suitable borrow material, preferably from the pond cut or from a nearby borrow area. The required volume of clayey soil is equal to the surface area of the pond interior times the liner thickness (measured perpendicular to the bottom and side slope surfaces). A large reserve volume is recommended to ensure that there is indeed sufficient clay volume after removing silt and sand pockets and other unsuitable materials.

5.9.6.3.2. Liner Thickness

Recommended minimum compacted clay liner thickness are 0.5 m on the pond bottom and 0.7 m on the side slopes, to allow for weathering, variations in actual thickness, pockets of poor quality material that escape detection, etc. If a clay core in the dyke is preferred over an upstream clay blanket liner, then the core should be well keyed into the bottom liner. A minimum core width of 3 m is suggested to allow economic and proper placement and compaction of the clay using large earth-moving equipment.

5.9.6.3.3. Hydraulic Conductivity of Compacted Clay

The in-situ hydraulic conductivity of the compacted clay liner should be predicted from laboratory tests on the proposed clay borrow material. Several samples should be selected representing the range of material within the designated borrow zone, not just the better material. Permeability tests should be performed on the samples compacted to the required density (i.e. 95% of standard Proctor maximum dry density) at a moisture content anticipated in the field. It is recommended that the sensitivity of the compacted clay hydraulic conductivity to variations in density and moisture content be determined. The designer must be prepared to ensure that the soil is brought to the specified moisture content (i.e. by wetting), unless the natural moisture content is already suitable.

A laboratory value for K should be calculated from the weighted average of the individual tests. The weighting of each test value should be according to the estimated percent of the borrow volume that the individual sample represents.

It is recommended that the liner design be based on a K in situ that is one order of magnitude larger than the average K (lab), i.e.: K (design) = K (in situ) = 10 x average K (lab)

The increase in the K value is a factor of safety to allow for the effects of macro-structure, poor quality borrow, etc., in the field. The K (design) and liner thickness values should meet the seepage criteria outlined in Section 5.9.6.2.6. If K (design) is too high, the more selective borrowing or adjustment of compaction moisture content could be investigated. Otherwise, an alternative liner material will be required.

Permeability tests shall be carried out on the soil material at each proposed stabilization basin site except in cases where the soil is unmistakably impervious. The permeability tests may take either of two forms:

1. Laboratory tests on samples from below the proposed bottom of the stabilization basin and from the material to be used in the dykes; and

2. Field seepage tests. These may be conducted in the following way. A pit shall be dug to the level of the proposed stabilization basin bottom and the bottom of the dug hole carefully cleaned. At least one test shall be conducted for every two hectares of stabilization basin area. A pipe with an internal diameter of at least 0.2 m and length of at least 1.2 m shall be carefully placed in a vertical position resting on the bottom of the hole. The hole shall be backfilled around the outside of the pipe to a height of 1.o m with carefully tamped soil. Particular care should be given to tamping of soil near the bottom.

The pipe shall be filled with water to a depth of 1.2 m. The water must be placed in the pipe gently so as not to disturb the soil at the bottom.

The drop in water level from a head of 1.2 m shall be recorded for each of at least three 24-hour periods, or until the readings become consistent. (Level shall be re-adjusted to 1.2 m at the beginning of each 24-hout period).

5.9.6.3.4. Subgrade Preparation

Clay should not be placed directly over gravel or other materials that do not provide an adequate filter to prevent piping erosion of the liner.

5.9.6.3.5. Liner Material Placement and Compaction

The clay should be placed in uniform, horizontal lifts of about 150 mm maximum loose thickness. The liner should be constructed in at least three lifts. Thin lifts ensure more uniform density, better bonding between lifts and reduces the likelihood of continuous seepage channels existing in the liner. Large lumps, cobbles and other undesirable materials are more easily identified in thin lifts. Lumps of soil greater than 100 mm in maximum dimension should be broken up prior to compaction. As far as practical, the liner should be built up in a uniform fashion over the pond area, in order to avoid sections of butted fill where seepage paths may develop.

Each lift should be compacted within the specified moisture content range to the required density using heavy, self-propelled sheepsfoot compactors. Lift surfaces that have been allowed to dry out should be scarified prior to placing of the next lift. Lift surfaces that have degraded due to precipitation etc., should either be removed or allowed to dry to the required moisture content and then be re-compacted. The completed liner should be smoothed out with a smooth-barrel compactor to reduce the liner surface area exposed to water absorption and swelling. The liner base should not be allowed to dry out or be exposed to freezing temperatures. Ideally, the liner should be flooded as soon as possible after construction and acceptance.

5.9.6.3.6. Construction Control

The most important form of quality control during construction of compacted clay liners will be observation and direction by the engineer. The characteristics of the desired liner material should be established in as much detail as possible (i.e. by colour, texture, moisture content, plasticity or characteristic features such as the mineralogy of pebbles in till). Quick visual or index test identification by experienced field personnel is probably the best way to detect poor quality material. An indirect but simple way of controlling liner quality is to perform frequent in situ density and moisture content tests. The density and moisture content may then be related to hydraulic conductivity by the relationships established during the laboratory test program (see Section 5.9.6.3.3). The frequency of tests should be increased when soil conditions are variable. The tests may be used to statistically evaluate the overall liner properties and to assess suspect zones in the liner.

In situ density and moisture content tests should be carried out on a routine basis for each lift. Tests should be conducted on a grid pattern (say 30 x 30 m to 60 x 60 m grids for large ponds and at closer spacing for small lagoons) and in suspect areas.

The completed liner may be assessed by performing in situ infiltration tests, which may be theoretically related to hydraulic conductivity values (see Section 5.9.6.3.3). It should be noted that the compacted clay liner is most likely to be partially saturated at the end of construction. The presence of 5 to 10 % air voids will result in an unsaturated K value that is somewhat higher than the saturated K value.

The completed liner may also be cored and the hydraulic conductivity of a trimmed sample can be tested in a suitable permeameter, i.e., odometer falling head tests or triaxial constant head tests. All holes created in the liner due to tests, stakes or other circumstances should be backfilled with well-compacted liner material.

5.9.6.3.7. Planning

The most important aspect of constructing a compacted clay liner may be the planning stage when the inspection engineer's role is defined, contract specifications are prepared and construction strategies are worked out. The engineer must have an adequate degree of control over material selection and methods of placement. The work procedure must be flexible with respect to earth movement.

Ideally, the borrow for a compacted clay liner would be the cut material just below the eventual pond invert. Thus, material may be cut and placed in a single operation for much of the pond liner area, although some stockpiling of borrow may be inevitable.

The lower lift of the liner might consist of reworked native soil broken up by tilling and re-compacted to eliminate fissures, etc. Nevertheless, the contract should allow for selective borrowing of cut material for liner use, for stockpiling, removal of undesirable materials and possible additional borrowing outside of the cut area.

5.9.6.4. Prefilling

Prefilling the pond should be considered in order to protect the liner, to prevent weed growth, to reduce odour and to maintain moisture content of the seal. However, the dykes must be completely prepared as described in Section 5.9.6.1.6 before the introduction of water.

5.9.6.5. Influent Lines

5.9.6.5.1. Material

Generally accepted material for underground sewer construction will be given consideration for the influent line to the lagoon. Unlined corrugated metal pipe should be avoided, however, due to corrosion problems. In material selection, consideration must be given to the quality of the wastes, exceptionally heavy external loadings, abrasion, soft foundations, and similar problems.

5.9.6.5.2. Manhole

A manhole or vented cleanout wye shall be installed prior to entrance of the influent line into the primary cell and shall be located as close to the dike as topography permits. Its invert shall be at least 150 mm above the maximum operating level of the lagoon and provide sufficient hydraulic pressure without surcharging the manhole.

5.9.6.5.3. Surcharging

The design and construction of influent piping shall insure that where surcharging exists, due to the head of the lagoon, no adverse effects will result. These effects shall include basement flooding and overtopping of manholes.

5.9.6.5.4. Forcemains

Forcemains terminating in a sewage lagoon should be fitted with a valve immediately upstream of the lagoon.

5.9.6.5.5. Flow Distribution

Flow distribution structures shall be designed to effectively split hydraulic and organic loads equally to primary cells.

5.9.6.5.6. Location

Influent lines shall be located along the bottom of the lagoon so that the top of the pipe is just below the average elevation of the lagoon seal, however, the pipe shall have adequate seal below it. The use of an exposed dyke to carry the influent line to the discharge points is prohibited.

5.9.6.5.7. Point of Discharge

The influent line to a square single celled lagoon should be essentially centre discharging. Each square cell of a multiple celled lagoon operated in parallel shall have its own near centre inlet but this does not apply to those cells following the primary cell, when series operation alone is used. Influent lines to single celled rectangular lagoons should terminate at approximately the third point farthest from the outlet structure. Influent and effluent piping should be located to minimize short-circuiting within the lagoon. Consideration should be given to multi-influent discharge points for primary cells of 5 ha or larger.

All aerated cells shall have influent lines, which distribute the load within the mixing zone of the aeration equipment. Consideration of multiple inlets should be closely evaluated for any diffused aeration system. For aerated lagoons the inlet pipe may go directly through the dyke and end at the toe of the inner slope.

5.9.6.5.8. Influent Discharge Apron

Inlet pipes should terminate with an upturned elbow, with the pipe extending 450 mm above the cell bottom. The end of the discharge line shall rest on a suitable concrete apron large enough to prevent the terminal influent velocity at the end of the apron from causing soil erosion. A minimum size apron of 1.0 m^2 shall be provided.

5.9.6.5.9. Pipe Size

The influent system shall be sized to permit peak raw sewage flow to be directed to any one of the primary cells. Influent piping should provide a minimum scouring velocity of 0.6 m/s.

5.9.6.6. Control Structures and Interconnecting Piping

5.9.6.6.1. Structure

Where possible, facilities design shall consider the use of multi-purpose control structures to facilitate normal operational functions such as drawdown and flow distribution, flow and depth measurement, sampling, pumps for re-circulation, chemical additions and mixing, and minimization of the number of construction sites within the dykes.

As a minimum, control structures shall be:

1. Accessible for maintenance and adjustment of controls;
2. Adequately ventilated for safety and to minimize corrosion;
3. Locked to discourage vandalism;
4. Contain controls to permit water level and flow rate control, complete shutoff, and complete draining;
5. Constructed of non-corrodible materials (metal-on-metal contact in controls should be of similar alloys to discourage electrochemical reactions); and
6. Located to minimize short-circuiting within the cell and avoid freezing and ice damage.

Recommended devices to regulate water level are valves, slide tubes, dual slide gates or effluent chambers complete with a water level regulating weir. Regulators should be designed so that they can be preset to stop flows at any pond elevation.

5.9.6.6.2. Piping

All piping shall be of ductile iron or other acceptable material. The piping shall not be located within or below the liner. Pipes should be anchored with adequate erosion control.

1. Drawdown Structure Piping:
   a) **Submerged Takeoffs** - For lagoons designed for shallow or variable depth operations, submerged take offs are recommended. Intakes shall be located a minimum of 3.0 m from the toe of the dyke and 0.6 m from the top of the seal, and shall employ vertical withdrawal;

   b) **Multi-Level Takeoffs** - For lagoons that are designed deep enough to permit stratification of lagoon content, multiple takeoffs are recommended. There shall be a minimum of 3 withdrawal pipes at different elevations. The bottom pipe shall conform to a submerged takeoff. The others should utilize horizontal entrance. Adequate structural support shall be provided.

c) **Surface Takeoffs** - For use under constant discharge conditions and/or relatively shallow lagoons under warm weather conditions, surface overflow-type withdrawal is recommended. Design should evaluate floating weir box or slide tube entrance with baffles for scum control.

d) **Maintenance Drawdown** - All lagoons shall have a lagoon drain to allow complete emptying, either by gravity or pumping, for maintenance. These should be incorporated into the above-described structures. In aerated lagoons where a diffused air aeration system and submerged air headers are used, provision should be made to drain each lagoon (independently of others) below the level of the air header.

e) **Emergency Overflow** - All cells shall be provided with an emergency overflow system, which overflows when the liquid reaches within 0.6 m of the top of the berms.

5.9.6.6.3. Hydraulic Capacity
The hydraulic capacity for continuous discharge structures and piping shall allow for at least the expected future peak sewage flows.

The hydraulic capacity for controlled-discharge systems shall permit transfer of water at a minimum rate of 150 mm of lagoon water depth per day at the available head.

5.9.6.6.4. Interconnecting Piping
Interconnecting piping for multiple unit installations operated in series should be valved or provided with other arrangements to regulate flow between structures and permit flexible depth control. The interconnecting pipe to the secondary cell should discharge horizontally near the lagoon bottom to minimize need for erosion control measures and should be located as near the dividing dyke as construction permits. Interconnection piping shall enable parallel or series flow patterns between cells.

5.9.6.6.5. Location
The outlet structure and the inter-connecting pipes should be located away from the corners where floating solids accumulate, and on the windward side to prevent short-circuiting.

5.9.7. Miscellaneous

5.9.7.1. Fencing
The lagoon area shall be enclosed with an adequate fence to prevent entering of livestock and discourage trespassing. Fencing should not obstruct vehicle traffic on top of the dyke. A vehicle access gate of sufficient width to accommodate mowing equipment shall be provided. All access gates shall be provided with locks.

5.9.7.2. Access

An all-weather access road shall be provided to the lagoon site to allow year-round maintenance of the facility.

5.9.7.3. Warning Signs

Appropriate permanent signs shall be provided along the fence around the lagoon to designate the nature of the facility and advise against trespassing. At least one sign shall be provided on each side of the site and one for every 150 m of its perimeter.

5.9.7.4. Flow Measurement

Provisions for flow measurement shall be provided on the outlet. Safe access to the device should be made to permit safe measurement.

5.9.7.5. Groundwater Monitoring

An approved system of wells or lysimeters may be required around the perimeter of the lagoon site to facilitate groundwater monitoring. The need for such monitoring will be determined on a case-by-case basis.

5.9.7.6. Pond Level Gauges

Pond level gauges shall be provided.

5.9.7.7. Service Building

A service building for laboratory and maintenance equipment shall be provided, if required.

5.9.7.8. Liquid Depth Operation

Optimum liquid depth is influenced to some extent by lagoon area since circulation in larger installations permits greater liquid depth. The basic plan of operation may also influence depth. Facilities to permit operation at selected depths between 0.6 to 1.5 m are recommended for operational flexibility. Where winter operation is desirable, the operating level can be lowered before ice formation and gradually increased to 1.5 m by the retention of winter flows. In the spring, the level can be lowered to any desired depth at the time surface runoff and dilution water are generally at a maximum. Shallow operation can be maintained during the spring with gradual increased depths to discourage emergent vegetation in the summer months. In the fall, the levels can be lowered and again be ready for retention of winter storage.

5.9.7.9. Pretreatment and Post-Treatment

The wastewater shall be treated by bar screens and grit removal before entering the lagoon. The treated effluent shall be disinfected, as per regulatory requirements, prior to discharging into the receiving water.

5.10. Kikuth Bioreactor (Phytoklare)

The Kikuth Bioreactor or Phytoklare wastewater treatment system (Root Zone or Reed Bed) is a process that can be used in areas where a conventional septic tank system cannot be used and in situations where a mechanical wastewater treatment system may not be practical or feasible.

The system is a proprietary one whose basic makeup consists of primary treatment (septic tank, settling tank), influent and effluent piping and a subsurface wetland treatment system consisting of one or more cells with an impermeable liner, specialized soil matrix, specific plants and a disinfection system if required.

The system is a passive system with no moving parts. Because the up front treatment consists of primary treatment only, then consideration has to be given to the impact that extraneous flows (e.g. wet weather) may have on the wetland component as excessive solids carry over could lead to blockage problems of the subsurface system. Also, this system may not be appropriate for areas where extreme cold temperatures are encountered.

5.11. BMS “Blivet”

The BMS “Blivet” is a patented package wastewater treatment system that incorporates a combination of technologies similar to the rotating biological contactor and conventional activated sludge and sedimentation processes. The treatment sections are composed of primary sedimentation utilizing lamella plates, aeration sections with rotating drums with a very large surface area inside (patented design). The effluent is drawn in via holes in its periphery. Once inside it passes through the maze of surfaces. The combined effect of being actively mixed with air and passing over the bacterial surfaces provides effective aerobic treatment. Then the effluent passes through a secondary settlement area using lamella plates and final discharge. Normal treatment standards are 20mg/L BOD and 30 mg/L SS. The manufacturer suggests that the process is particularly suitable for population equivalents of between 10 to 3000 residents. They are used in areas not connected to mains sewers such as hotels, golf clubs, country clubs, holiday resorts, housing developments, condominiums, small townships and military and civil institutions.

5.12. Biogreen

The Biogreen system is designed primarily to provide wastewater treatment for individual homes or small communities and is comprised of seven chambers. The first two chambers are settling tanks, which remove sediment, debris and floating material from the influent. The first settling tank is larger than conventional systems and acts as a buffer for controlling the organic load through the unit. This is often a serious problem in conventional systems but is easily dealt with in a Biogreen system. The water passes from the settling tanks to the fermentation tank, which is filled with filters, and most of the anaerobic digestion of the sewage occurs here.

The next stage of the treatment process is aeration. The flow from the fermentation tank is controlled by a control valve to the first of two aeration tanks. This minimizes the shock loading which may occur in the aeration chamber. A draft tube is located in the middle of each aeration chamber, into which air is injected. This creates ideal circulation and maintains an adequate DO concentration. A DO gradient from zero to saturation is created, which allows for a longer food chain for nutrient digestion. This reduces sludge production as compared to existing methods. The final step is the effluent holding tank, which removes suspended solids before discharge.

5.13. Other Biological Systems

New biological treatment schemes with promising applicability in wastewater treatment may be considered if the required engineering data for new process evaluation is provided in accordance with Section 5.4.3.2. A number of new biological systems are described below. These systems typically are manufactured by companies who hold proprietary designs and as proprietary information cannot be included in this manual the design data presented is fairly general in nature. A description of these systems mainly describing their application and typical loading rates is provided here. New treatment schemes may be added to the main section of this chapter when sufficient and adequate design data becomes available. These additions will be noted in the revision record.

5.13.1. Biological Aerated Filters

Biological aerated filters (BAFs) are submerged, granular media upflow filters, which treat wastewater by biologically converting carbonaceous and nitrogenous matter using biomass fixed to the media and physically capturing suspended solids within the media. The filters are aerated to remove carbonaceous matter and convert ammonia-nitrogen to nitrates via nitrification. Non-aerated filters in the presence of supplemental carbonaceous organic matter can convert nitrates to nitrogen gas through denitrification. BAFs are designed either as co-current backwash or counter current backwash systems. The co-current backwash design has a nozzle deck supporting a granular media that has a specific gravity greater than 1.0. Pre-treated wastewater is introduced under the nozzle deck and flows up through a slightly expanded media bed, and effluent leaves the filter from above the media. Process air is introduced just above the nozzle deck (the bed is not aerated for denitrification). During backwash, wash water and air scour are introduced below the nozzle deck and flow up through the bed. Wash water is pumped to the head of the plant or directly to solids handling.

The counter current backwash BAF operates under the same general principles, except that the granular media has a specific gravity less than 1.0. Therefore, the media float and are retained from above by the nozzle deck. During backwash, wash water flows by gravity through the media. Process air is introduced below the media; therefore, scour air moves counter current to the wash water flow.

5.13.1.1. Design Features

The granular media bed for both designs typically is 3 to 4 m deep and the media are 3 to 6 mm in diameter. The media-specific surface area ranges from 500 to 2000 m^2/m^3. The contact time in the media typically is 0.5 to 1.0 hour. The media bed is backwashed every 24 to 48 hours for 20 to 40 minutes using a wash water volume about three times the media volume. Backwash water from a single event is collected in a storage tank and returned to the head of the plant or directly to solids processing over a 1 to 2 hour period. Backwash water typically contains from 400 to 1,200 mg/L of suspended solids. The backwash water recycle flow can represent up to 20% of the raw influent wastewater flow. Most manufacturers have estimated that solids production from the BAF system is comparable to that of a conventional activated sludge system. Effluent pollutant concentrations from a single BAF cell increases for approximately 30 minutes following a backwash event, so a minimum of four cells should be included in any design to dampen these spikes.

The nozzle deck features polyethylene nozzles that prevent media loss and assist in evenly distributing flow across the bed. The reported media loss from the BAF system is less than 2% per year. The nozzle openings are slightly smaller than the media and require that influent be pre-treated with a fine screen to prevent plugging. Headloss across the media bed can be more than 2 m prior to backwash. In existing installations, the filters are constructed above grade. The combination of the tall structure (6 m) and headloss across the bed requires pumping influent flow to the BAF in most situations. In addition, the co-current designs require pumping of wash water, which is a significant, but intermittent, energy demand.

Process air is required in BAF cells that are removing carbonaceous organic matter (biochemical oxygen demand or BOD) and are nitrifying ammonia-nitrogen. The process aeration system consists of coarse to medium bubble diffusers on a stainless steel piping grid. Because of the difficulty in accessing the aeration grid, the diffusers are constructed as simply and reliably as possible. The amount of air that must be added to the system is determined by the oxygen demand of the biomass. Energy for process air can represent more than 80 % of the energy demand in a BAF system.

5.13.1.2. Configurations

BAFs can operate in different process configurations, depending on the facilities, effluent goals, and wastewater characteristics. The process can follow either chemically assisted primary sedimentation or an activated sludge system. This level of treatment is required because of a BAF system's sensitivity to high influent BOD and suspended solids loadings. Following primary sedimentation, BAF cells can be operated for carbonaceous BOD removal or, under lower loading rates (less than 1.5 kg BOD/m^{3}d), for both carbonaceous BOD and ammonia-nitrogen removal. A cell can operate in a nitrification mode following an activated sludge system or another BAF cell removing carbonaceous BOD. A denitrification BAF process can follow either an activated sludge or BAF system that is nitrifying.

5.13.1.3. Performance

The performance of BAFs in terms of allowable loading rates and effluent quality depends on influent wastewater quality and temperature. In general, higher organic or suspended solids influent loadings result in higher effluent concentrations. Adequate water velocity is necessary to provide scouring of the biomass and even flow distribution across the media bed. Inadequate water velocity can result in premature bed plugging; this is especially true for denitrification reactors in which the effects of air scouring are not present.

Factors that positively affect complete nitrification include:
1. Warm water temperature;
2. Adequate aeration and good air distribution, and
3. Low carbonaceous BOD and suspended solids loading.

Denitrification usually requires methanol addition, and water velocities must be greater than 10 m/hr.

5.13.2. Moving Bed Biofilm Reactors

The Norwegian company Kaldnes Miløteknologi (KMT) developed the patented MBBR process. The basic concept of the MBBR is to have continuously operating, non-cloggable biofilm reactors with no need for backwashing or return sludge flows, low head-loss and high specific biofilm surface area. This is achieved by having the biomass grow on small carrier elements that move along with the water in the reactor. The movement is normally caused by coarse-bubble aeration in the aeration zone and mechanical mixing in an anoxic/anaerobic zone. However, for small plants, mechanical mixers are omitted for simplicity reasons and pulse aeration for a few seconds a few times per day can be used to move the biofilm carriers in anoxic reactors.

The biofilm carrier elements are made of 0.96 specific gravity polyethylene and shaped like small cylinders, with a cross in the inside of the cylinder and longitudinal fins on the outside. To keep the biofilm elements in the reactor, a screen of perforated places is placed at the outlet of the reactor. Agitation constantly moves the carrier elements over the surface of the screen; the scrubbing action prevents clogging. Almost any size or shape tank can be retrofitted with the MBBR process. The filling of carrier elements in the reactor may be decided for each case, based on degree of treatment desired, organic and hydraulic loading, temperature and oxygen transfer capability. The reactor volume is totally mixed and consequently there is no "dead" space or unused space in the reactor. Organic loading rates for these reactors are typically in the order of 3.5 – 7.0 g BOD/m^2 of media surface area/d for BOD removal and less than 3.5 g BOD/m^2 of media surface area/d for nitrification.

5.13.3. Membrane Bioreactors

Membrane Bioreactors consist of a suspended growth biological reactor (activated sludge system variation) integrated with a microfiltration membrane system. The key to the technology is the membrane separator, which allows elevated levels of biomass to degrade or remove the soluble form of the organic pollutants from the waste stream. These systems typically operate in the nanofiltration or microfiltration range, which results in removal of particles greater than 0.01 and 0.1 µm, respectively.

5.13.3.1. Configuration

Membrane bioreactors can be configured in a number of different ways, however, the two main configurations differ by those in which the membranes are submersed directly in the bioreactor and those which contain external membrane process tankage. When membrane modules are submersed into the bioreactor, they are in direct contact with the wastewater and sludge. A vacuum is created within the hollow fibres by the suction of a permeate pump.

The treated water passes through the membrane, enters the hollow fibres and is pumped out by the permeate pump. An airflow may be introduced to the bottom of the membrane module to create turbulence which scrubs and cleans the membrane fibres keeping them functioning at a high flux rate. The filtrate or permeate is then collected for reuse or discharge. Outboard membrane processes operate in a similar manner however, the membranes are contained in a separate tank through which the wastewater requiring filtration constantly flows. Again air is often added for both treatment and membrane scouring purposes. The main difference between the two configurations lies in the membrane cleaning processes where membranes submersed within the aeration tanks must be removed for cleaning while outboard membranes are cleaned by evacuating the membrane tankage and providing for equalization during the cleaning procedures within the main aeration tank.

5.13.3.2. Process Description

The benefits of these processes are consistent effluent quality, reduced footprint, increased expansion capabilities within the same tankage, and ease of operation. Tertiary quality effluent is the normal output of a membrane bioreactor. Typical effluent quality is presented in Table 5.21. Virtually no solids are lost via the permeate stream and the wasting of solids is reduced. As a result, the sludge age can be very accurately determined.

Nitrification for ammonia removal is easily achieved by optimizing reactor and sludge age to specific wastewater characteristics and effluent requirements. Absolute control of the nitrifiers results in high nitrification rates even in winter periods and under adverse and unstable conditions. If required, denitrification can be achieved with for membrane processes as when operating at a MLSS of 15,000 mg/L and higher, the mixed liquor rapidly becomes anoxic in the absence of a continuous stream of air. Furthermore, the high levels of biomass ensure that in the anoxic zone, at all times there are enough denitrifiers to efficiently convert the nitrates into nitrogen gas.

Table 5.21
Membrane Bioreactor Effluent Quality

Parameters	Secondary Treatment	Tertiary Treatment	Membrane Bioreactor
BOD (mg/L)	10-12	< 5	< 2
TSS (mg/L)	10-15	< 1	< 1
NH_3 (mg/L)	1-10	1-10	< 0.3
Total P (mg/L)	> 1	0.1-0.5	< 0.1
Total Coliforms (MPN/100 mL)	> 1000	> 1000	< 100
Fecal Coliforms (MPN/100 mL)	> 100	> 100	< 10
Sludge Yields (kg/kg BOD_5 removed)	0.3-0.6	0.3-0.6	0.1-0.3

5.13.4. Small Treatment Plants

Small treatment plants generally utilize the biological treatment process and are installed as a single "package" unit. The package treatment plant is usually purchased either complete with steel tankage or with the plant equipment ready for installation in "poured in place" concrete tankage.

The process design shall be in accordance with the guidelines presented in the previous sections.

Simplicity of operation and durability are essential in the design of a small treatment plant, which is quite often in a remote location and frequently only receives minimal operator attention.

It is essential that the treatment plant supplier or his representative provide complete operator training during the plant start-up period.

A comprehensive operating manual shall be provided. The manual shall include such items as follows:

1. Process operation, including flow diagrams and comprehensive troubleshooting and remedial action list;
2. Maintenance schedule, including frequency of attention required, e.g. daily, weekly, etc.;
3. Comprehensive equipment information, including drawings, serial numbers, part numbers, etc.; and
4. Safety precautions and action required in the event of an emergency.

5.13.4.1. Flows

When insufficient data is available, the values listed in Table 5.1, may be used in computing the design flow.

5.14. Nutrient Removal and Tertiary Treatment

5.14.1. Phosphorous Removal

5.14.1.1. Applicability

The following factors should be considered when determining the need for phosphorus control at municipal wastewater treatment facilities:

1. The present and future phosphorus loadings from the existing municipal wastewater treatment facility to the receiving water;

2. The CCME Ambient Water Quality Guidelines

3. The existing phosphorus levels in the receiving water and the effects of these levels on the rate of eutrophication along the entire length of receiving waters;

4. The predicted response of the receiving water to increased phosphorus loadings;

5. The existing and desired water quality of the receiving water along its entire length;

6. The existing and projected uses of the receiving water; and

7. Consideration of the best practicable technology available to control phosphorus discharges.

5.14.1.2. Phosphorus Removal Criteria

A municipal wastewater treatment facility shall be required to control the discharge of phosphorus if the following conditions exist:

1. Eutrophication of the receiving water environment is either occurring or may occur at a rate which may affect the existing and potential uses of the water environment; or

2. The municipal wastewater effluent discharge is contributing or may contribute significantly to the rate of receiving water eutrophication.

5.14.1.3. Method of Removal

Acceptable methods for phosphorus removal shall include chemical precipitation, high rate filtration or biological processes.

5.14.1.4. Design Basis

5.14.1.4.1. Preliminary Testing

Laboratory, pilot or full scale studies of various chemical feed systems and treatment processes are recommended for existing plant facilities to determine the achievable performance level, cost-effective design criteria, and ranges of required chemical dosages.

The selection of a treatment process and chemical dosage for a new facility should be based on such factors as influent wastewater characteristics, effluent requirements, and anticipated treatment efficiency.

5.14.1.4.2. System Flexibility

Systems shall be designed with sufficient flexibility to allow for several operational adjustments in chemical feed location, chemical feed rates, and for feeding alternate chemical compounds.

5.14.1.5. Effluent Requirements

If phosphorus control is required, the maximum acceptable concentration of final effluent phosphorus and/or the maximum acceptable mass loading to the receiving stream shall be established on a site-specific basis.

5.14.1.6. Process Requirements

5.14.1.6.1. **Dosage**

Typical chemical dosage requirements of various chemicals required for phosphorus removal are outlined in Table 5.22.

Dosages will vary with the phosphorus concentration in the effluent. The required chemical dosage shall include the amount needed to react with the phosphorus in the wastewater, the amount required to drive the chemical reaction to the desired state of completion, and the amount required due to inefficiencies in mixing or dispersion. Excessive chemical dosage should be avoided.

Table 5.22
Typical Chemical Dosage Requirements for Phosphorus Removal

Type of Treatment	Addition Point	Dosage Rate (mg/L)		
Plant		**Chemical**	**Range**	**Average**
Mechanical:				
Primary	Raw Sewage	Alum Ferric Chloride Lime	100 6 – 30 167 - 200	100 16 185
Secondary	Raw Sewage	Lime Alum Ferric Chloride	- 40 – 100 -	- 70 -
	Secondary Section	Lime Alum Ferric Chloride	- 30 – 150 2 - 30	- 65 11
Lagoons:				
Seasonal Retention Lagoons	Batch Dosage to Cells	Alum Ferric Chloride Lime	100 – 210 17 – 22 250 - 350	163 20 300
Continuous Discharge Lagoons	Raw Sewage	Alum Ferric Chloride Lime	225 20 400	225 20 400

5.14.1.6.2. Chemical Selection

The choice of lime or the salts of aluminum or iron should be based on the wastewater characteristics and the economics of the total system.

When lime is used it may be necessary to neutralize the high pH prior to subsequent treatment in secondary biological systems or prior to discharge in those flow schemes where lime treatment is the final step in the treatment process.

5.14.1.6.3. Chemical Feed System

In designing the chemical feed system for phosphorus removal, the following points should be considered:

1. The need to select chemical feed pumps, storage tanks and piping suitable for use with the chosen chemicals(s);

2. Selection of chemical feed equipment with the required range in capacity;

3. The need for a standby chemical feed pump;

4. Provision of flow pacing for chemical pumps proportional to sewage flow rates;

5. Flexibility by providing a number of chemical application points;

6. The need for protection of storage and piping from the effect of low temperatures;

7. Selection of the proper chemical storage volume;

8. The need for ventilation in chemical handling rooms; and

9. Provision for containment of any chemical spills.

5.14.1.6.4. Chemical Feed Points

Selection of chemical feed points shall include consideration of the chemicals used in the process, necessary reaction times between chemical and polyelectrolyte additions, and the wastewater treatment processes and components utilized.

Considerable flexibility in feed location should be provided, and multiple feed points are recommended.

5.14.1.6.5. Flash Mixing

Each chemical must be mixed rapidly and uniformly with the flow stream. Where separate mixing basins are provided, they should be equipped with mechanical mixing devices. The detention period should be at least 30 seconds.

5.14.1.6.6. Flocculation

The particle size of the precipitate formed by chemical treatment may be very small. Consideration should be given in the process design to the addition of synthetic polyelectrolytes to aid settling. The flocculation equipment should be adjustable in order to obtain optimum floc growth, control deposition of solids, and prevent floc destruction.

5.14.1.6.7. Liquid - Solids Separation

The velocity through pipes or conduits from flocculation basins to settling basins should not exceed 0.5 m/s in order to minimize floc destruction. Entrance works to settling basins should also be designed to minimize floc shear.

Settling basin design shall be in accordance with criteria outlined in Section 5.6.

For design of the sludge handling system, special consideration should be given to the type and volume of sludge generated in the phosphorus removal process.

5.14.1.6.8. **Filtration**

Effluent filtration shall be considered where effluent phosphorus concentrations of less than 1 mg/L must be achieved.

5.14.1.7. Feed Systems

5.14.1.7.1. **Location**

All liquid chemical mixing and feed installations should be installed on corrosion resistant pedestals and elevated above the highest liquid level anticipated during emergency conditions.

Lime feed equipment should be located so as to minimize the length of slurry conduits. All slurry conduits shall be accessible for cleaning.

5.14.1.7.2. Liquid Chemical Feed System

Liquid chemical feed pumps should be of the positive displacement type with variable feed rate. Pumps shall be selected to feed the full range of chemical quantities required for the phosphorus mass loading conditions anticipated with the largest unit out of service.

Screens and valves shall be provided on the chemical feed pump suction lines.

An air break or anti-siphon device shall be provided where the chemical solution stream discharges to the transport water stream to prevent an induction effect resulting in overfeed.

Consideration shall be given to providing pacing equipment to optimize chemical feed rates.

5.14.1.7.3. Dry Chemical Feed System

Each dry chemical feeder shall be equipped with a dissolver, which is capable of providing a minimum 5-minute retention at the maximum feed rate.

Polyelectrolyte feed installations should be equipped with two solution vessels and transfer piping for solution make-up and daily operation.

Make-up tanks shall be provided with an educator funnel or other appropriate arrangement for wetting the polymer during the preparation of the stock feed solution. Adequate mixing should be provided by a large-diameter low-speed mixer.

5.14.1.8. Storage Facilities

5.14.1.8.1. **Size**

Storage facilities shall be sufficient to insure that an adequate supply of the chemical is available at all times. The exact size required will depend on the size of the shipment, length of delivery time, and process requirements. Storage for a minimum of 10-days supply should be provided.

***5.14.1.8.2.* Location**

The liquid chemical storage tanks and tank fill connections shall be located within a containment structure having a capacity exceeding the total volume of all storage vessels. Valves on discharge lines shall be located adjacent to the storage tank and within the containment structure.

Auxiliary facilities, including pumps and controls, within the containment area shall be located above the highest anticipated liquid level. Containment areas shall be sloped to a sump area and shall not contain floor drains.

Bag storage should be located near the solution make-up point to avoid unnecessary transportation and housekeeping problems.

***5.14.1.8.3.* Accessories**

Platforms, ladders, and railings should be provided as necessary to afford convenient and safe access to all filling connections, storage tank entries, and measuring devices.

Storage tanks shall have reasonable access provided to facilitate cleaning.

5.14.1.9. Other Requirements

***5.14.1.9.1.* Materials**

All chemical feed equipment and storage facilities shall be constructed of materials resistant to chemical attack by all chemicals normally used for phosphorus treatment.

5.14.1.9.2. Temperature, Humidity and Dust Control

Precautions shall be taken to prevent chemical storage tanks and feed lines from reaching temperatures likely to result in freezing or chemical crystallization at the concentrations employed. A heated enclosure or insulation may be required.

Consideration should be given to temperature, humidity and dust control in all chemical feed room areas.

***5.14.1.9.3.* Cleaning**

Consideration shall be given to the accessibility of piping. Piping should be installed with plugged wyes, tees or crosses at changes in direction to facilitate cleaning.

5.14.1.9.4. Drains and Drawoff

Above-bottom drawoff from chemical storage or feed tanks shall be provided to avoid withdrawal of settled solids into the feed system. A bottom drain shall also be installed for periodic removal of accumulated settled solids. Provisions shall be made in the fill lines to prevent back siphonage of chemical tank contents.

5.14.1.10. Hazardous Chemical Handling

The requirements of Occupational Health and Safety, and Hazardous Chemical Handling shall be met.

5.14.1.11. Sludge Handling and Dewatering

Consideration shall be given to the type and additional capacity of the sludge handling facilities needed when chemicals are added.

Design of dewatering systems should be based, where possible, on an analysis of the characteristics of the sludge to be handled. Consideration should be given to the ease of operation, effect of recycle streams generated, production rate, moisture content, dewater ability, final disposal, and operating cost.

5.14.2. Ammonia Removal

5.14.2.1. Breakpoint Chlorination

5.14.2.1.1. Applicability

The breakpoint chlorination process is best suited for removing relatively small quantities of ammonia, less than 5 mg/L NH_3-N, and in situations whose low residuals of ammonia or total nitrogen are required.

5.14.2.1.2. Design Considerations

5.14.2.1.2.1. Mixing

The reaction between ammonia and chlorine occurs instantaneously, and no special design features are necessary except to provide for complete uniform mixing of the chlorine with the wastewater. Good mixing can best be accomplished with in-line mixers or backmixed reactors. A minimum contact time of 10 minutes is recommended.

5.14.2.1.2.2. Dosage

The sizing of the chlorine producing and/or feed device is dependent on the influent ammonia concentration to be treated as well as the degree of pre-treatment the wastewater has received. As the level of wastewater pretreatment increases, the required amount of chlorine decreases and approaches the theoretical amount required to oxidize ammonia to nitrogen (7.6 mg/L Cl_2:1 mg/L NH_3-N). Table 5.23 shows the quantities of chlorine required, based on operating experience as well as recommended design capabilities. These ratios are applied to the maximum anticipated influent ammonia concentration.

5.14.2.1.2.3. Monitoring

If insufficient chlorine is available to reach the breakpoint, no nitrogen will be formed and the chloramines formed ultimately will revert back to ammonia.

Provisions should be made to continuously monitor the waste, following chlorine addition, for free chlorine residual and to pace the chlorine feed device to maintain a set-point free chlorine residual.

Table 5.23
Quantities of Chlorine Required for Three Wastewater Sources

Wastewater Source	Chlorine: NH_3 – N Ratio to Reach Breakpoint	
	Experience	Recommended Design Capability
Raw	10:1	13:1
Secondary Effluent	9:1	12:1
Lime Settled and Filtered Secondary Effluent	8:1	10:1

5.14.2.1.2.4. Standby Equipment
The chemical feed assembly used for ammonia removal by breakpoint chlorination is considered in the preliminary design of the complete chlorination system, including those requirements for pre-chlorination, intermediate, and post-chlorination applications. Depending on the use of continuous chlorination at points within the system, some consideration is given to the use of standby chlorination equipment for the ammonia removal system. Reliability needs and maximum dosage requirements for the various application points shall also be examined when sizing the equipment.

5.14.2.1.2.5. pH Adjustment
Except for wastewaters having a high alkalinity or treatment systems employing lime coagulation prior to chlorination, provisions shall be made to feed an alkaline chemical to keep the pH of the wastewater in the proper range. A method for measuring and pacing the alkaline chemical feed pump to keep the pH in the desired range also should be provided.

5.14.2.2. Air Stripping
5.14.2.2.1. Applicability
The ammonia air stripping process is most economical if it is preceded by lime coagulation and settling. The ammonia stripping process can be used in a treatment system employing biological treatment or in a physical-chemical process. In most instances, more than 90% of the nitrogen in raw domestic wastewater is in the form of ammonia, and the ammonia stripping process can be readily applied to most physical-chemical treatment systems. However, when the ammonia stripping process is to be preceded by a biological process, care must be exercised to insure that nitrification does not occur in the secondary treatment process.

There is one serious limitation of the ammonia stripping process that should be recognized; namely, it is impossible to operate a stripping tower at air temperatures less than 0°C because of freezing within the tower. For treatment plants in cold weather locations, high pH stripping ponds may provide a simple solution to the problem of nitrogen removal.

5.14.2.2.2. Design Considerations

5.14.2.2.2.1. Tower Packing

Packings used in ammonia stripping towers may include 10 by 40 mm wood slats, plastic pipe, and a polypropylene grid. No specific packing spacing has been established. Generally, the individual splash should be spaced 40 to 100 mm horizontally and 50 to 100 mm vertically. A tighter spacing is used to achieve higher levels of ammonia removal and a more opening spacing is used where lower levels of ammonia removal are acceptable. Because of the large volume of air required, towers should be designed for a total air headloss of less than 50 to 75 mm of water. Packing depths of 6 to 7.5 m should be used to minimize power costs.

5.14.2.2.2.2. Hydraulic Loadings

Allowable hydraulic loading is dependent on the type and spacing of the individual splash bars. Although hydraulic loading rates used in ammonia stripping towers should range from 0.7 to 2.0 L/m^2s removal efficiency is significantly decreased at loadings in excess of 1.3 L/m^2s. The hydraulic loading rate should be such that a water droplet is formed at each individual splash bar as the liquid passes through the tower.

5.14.2.2.2.3. Air Requirements

Air requirements vary from 2200 to 3800 L/s for each L/s being treated in the tower. The 6 to 7.5 m of tower packing will normally produce a pressure drop of 15 to 40 mm of water.

5.14.2.2.2.4. Temperature

Air and liquid temperatures have a significant effect on the design of an ammonia-stripping tower. Minimum operating air temperature and associated air density should be considered when sizing the fans to meet the desired air supply. Liquid temperature also affects the level of ammonia removal.

5.14.2.2.2.5. General Construction Features

The stripping tower may be either of the counter-current (air inlet at base) or cross flow (air inlet along entire depth of fill) type. Generally, provisions should be made to have the capability to recycle tower effluent to increase the removal of ammonia nitrogen during cooler temperatures. Provisions shall be made in the design of the tower structure and fill so that the tower packing is readily accessible or removable for removing possible deposits of calcium carbonate.

5.14.2.2.2.6. Process Control

During periods of tower operation when temperature, air and wastewater flow rates, and scale formation are under control, the major process requirement necessary to insure satisfactory ammonia removal is to control the influent pH. pH control should be practiced in the upstream lime-coagulation-settling process.

This basin should be monitored closely to prevent excessive carryover of lime solids into the ammonia stripping process. Normal lime-addition required to raise the pH to 11.5 is 300 to 400 mg/L (as CaO).

5.14.3. Biological Nutrient Removal

5.14.3.1. Biological Phosphorus Removal

A number of biological phosphorus removal processes exist that have been developed as alternatives to chemical treatment. Phosphorus is removed in biological treatment by means of incorporating orthophosphate, polyphosphate, and organically bound phosphorus into cell tissue. The key to the biological phosphorus removal is the exposure of the microorganisms to alternating anaerobic and aerobic conditions. Exposure to alternating conditions stresses the microorganisms so that their uptake of phosphorus is above normal levels.

Phosphorus is not only used for cell maintenance, synthesis, and energy transport but is also stored for subsequent use by the microorganisms. The sludge containing the excess phosphorus is either wasted or removed through a sidestream to release the excess. The alternating exposure to anaerobic and aerobic conditions can be accomplished in the main biological treatment process, or "mainstream," or in the return sludge stream, or "sidestream."

5.14.3.1.1. Mainstream Phosphorus Removal (A/O Process)

The proprietary A/O process is a single sludge suspended-growth system that combines anaerobic and aerobic sections in sequence. Settled sludge is returned to the influent end of the reactor and mixed with the incoming wastewater. Under anaerobic conditions, the phosphorus contained in the wastewater and the recycled cell mass is released as soluble phosphates. Some BOD reduction also occurs in this stage. The phosphorus is then taken up by the cell mass in the aerobic zone. Phosphorus is removed from the liquid stream in the waste activated sludge. The concentration of phosphorus in the effluent is dependent mainly on the ratio of BOD to phosphorus of the wastewater treated.

5.14.3.1.2. Sidestream Phosphorus Removal (PhoStrip Process)

In the proprietary PhoStrip Process a portion of the return activated sludge from the biological treatment process is diverted to an anaerobic phosphorus-stripping tank. The retention time in the stripping tank typically ranges from 8 to 12 hours.

The phosphorus released in the stripping tank passes out of the tank in the supernatant, and the phosphorus-poor activated sludge is returned to the aeration tank. The phosphorus-rich supernatant is treated with lime or another coagulant in a separate tank and discharged to the primary sedimentation tanks or to a separate flocculation/clarification tank for solids separation. Phosphorus is removed from the system in the chemical precipitant. Conservatively designed PhoStrip and associated activated-sludge systems are capable of consistently producing an effluent with a total phosphorus content of less than 1.5 mg/L before filtration.

5.14.3.1.3. Design Criteria

Table 5.24 provides various design criteria with regards to biological phosphorous removal processes.

Table 5.24
Design Criteria for Biological Phosphorus Removal

Design Parameter	**Treatment Process**		
	A/O	**PhoStrip**	SBR
Food/Microorganism Ratio (kg BOD_5/kg MLVSS.d)	0.2 – 0.7	0.1 – 0.5	0.15 – 0.5
Solids Retention Time (day)	2 - 25	10 – 30	
MLSS (mg/L)	2000 – 4000	600 - 5000	2000 – 3000
Hydraulic Retention Time (hours)			
Anaerobic Zone	0.5 – 1.5	8 – 12	1.8 – 3
Aerobic Zone	1 - 3	4 - 10	1.0 – 4
Return Activated Sludge (% of Influent Flowrate)	25 - 40	20 - 50	N/A
Stripper Underflow (% of Influent Flowrate)	N/A	10 - 20	N/A

5.14.3.2. Biological Nitrogen Removal

The principal nitrogen conversion and removal processes are conversion of ammonia nitrogen to nitrate by biological nitrification and removal of nitrogen by biological nitrification/denitrification.

5.14.3.2.1. Nitrification

Biological nitrification consists of the conversion of ammonia nitrogen to nitrite followed by the conversion of nitrite to nitrate. This process does not increase the removal of nitrogen from the waste stream over that achieved by conventional biological treatment. The principal effect is that nitrified effluent can be denitrified biologically. To achieve nitrification, all that is required is the maintenance of conditions suitable for the growth of nitrifying organisms.

Nitrification is also used when treatment requirements call for oxidation of ammonia-nitrogen. Nitrification may be carried out in conjunction with secondary treatment or in a tertiary stage. In each case, either suspended growth or attached growth reactors can be used.

Table 5.25 provides design criteria for various nitrification processes.

Table 5.25
Design Criteria for Nitrification

Design Parameter	Single Stage	Separate Storage
Food/Microorganism Ratio (kg Bod5/kg MLVSS.day)	0.12 – 0.25	0.05 – 0.2
Solids Retention Time (day)	8 – 20	15 – 100
MLSS (mg/L)	1500 – 3500	1500 – 3500
Hydrauic Retention Time (hours)	6 – 15	3 – 6
Return Activated Sludge (% of Influent Flowrate)	50 - 150	50 - 200

5.14.3.2.2. Combined Nitrification/Denitrification

The removal of nitrogen by biological nitrification/denitrification is a two-step process. In the first step, ammonia is converted aerobically to nitrate (NO3-) (nitrification). In the second step, nitrates are converted to nitrogen gas (denitrification).

The removal of nitrate by conversion to nitrogen gas can be accomplished biologically under anoxic conditions. The carbon requirements may be provided by internal sources, such as wastewater and cell material, or by an external source.

5.14.3.2.2.1. Bardenpho Process (Four-Stage)

The four-stage proprietary Bardenpho process uses both the carbon in the untreated wastewater and carbon from endogenous decay to achieve denitrification. Separate reaction zones are used for carbon oxidation and anoxic denitrification. The wastewater initially enters an anoxic denitrification zone to which nitrified mixed liquor is recycled from a subsequent combined carbon oxidation nitrification compartment. The carbon present in the wastewater is used to denitrify the recycled nitrate. Because the organic loading is high, denitrification proceeds rapidly. The ammonia in the wastewater passes unchanged through the first anoxic basin to be nitrified in the first aeration basin.

The nitrified mixed liquor from the first aeration basin passes into a second anoxic zone, where additional denitrification occurs using the endogenous carbon source. The second aerobic zone is relatively small and is used mainly to strip entrained nitrogen gas prior to clarification. Ammonia released from the sludge in the second anoxic zone is also nitrified in the last aerobic zone.

5.14.3.2.2.2. Oxidation Ditch

In an oxidation ditch, mixed liquor flows around a loop-type channel, driven and aerated by mechanical aeration devices. For nitrification/denitrification applications, an aerobic zone is established immediately downstream of the aerator, and an anoxic zone is created upstream of the aerator. By discharging the influent wastewater stream at the upstream end of the anoxic zone, some of the wastewater carbon source is used for denitrification. The effluent from the reactor is taken from the end of the aerobic zone for clarification. Because the system has only one anoxic zone, nitrogen removals are lower than those of the Bardenpho process.

5.14.3.3. Combined Biological Nitrogen and Phosphorus Removal

A number of biological processes have been developed for the combined removal of nitrogen and phosphorus. Many of these are proprietary and use a form of the activated sludge process but employ combinations of anaerobic, anoxic, and aerobic zones or compartments to accomplish nitrogen and phosphorus removal.

5.14.3.3.1. A^2/O Process

The proprietary A^2/O process provides an anoxic zone for denitrification with a detention period of approximately one hour. The anoxic zone is deficient in dissolved oxygen, but chemically bound oxygen ion the form of nitrate or nitrite is introduced by recycling nitrified mixed liquor from the aerobic section. Effluent phosphorus concentrations of less than 2 mg/L can be expected without effluent filtration; with effluent filtration, effluent phosphorus concentrations may be less than 1.5 mg/L.

5.14.3.3.2. Bardenpho Process (Five-Stage)

The proprietary Bardenpho process can be modified for combined nitrogen and phosphorus removal. The Phoredox modification of the bardenpho process incorporates a fifth (anaerobic) stage for phosphorus removal. The five-stage system provides anaerobic, anoxic, and aerobic stages for phosphorus, nitrogen, and carbon removal. A second anoxic stage is provided for additional denitrification using nitrate produced in the aerobic stage as the electron acceptor and the endogenous organic carbon as the electron donor. The final aerobic stage is used to strip residual nitrogen gas from solution and to minimize the release of phosphorus in the final clarifier. Mixed liquor from the first aerobic zone is recycled to the anoxic zone.

5.14.3.3.3. **UCT Process**

The UCT process eliminates return activated sludge to the anoxic stage and the internal recycle is from the anoxic stage to the anaerobic stage. By returning the activated sludge to the anoxic stage, the introduction of nitrate to the anaerobic stage is eliminated, thereby improving the release of phosphorus in the anaerobic stage. The internal recycle feature provides for increased organic utilization in the anaerobic stage. The mixed liquor from the anoxic stage contains substantial soluble BOD but little nitrate. The recycle of the anoxic mixed liquor provides for optimal conditions for fermentation uptake in the anaerobic stage.

5.14.3.3.4. Design Criteria

Table 5.26 provides design criteria for various combined biological nitrogen and phosphorous removal processes.

Table 5.26
Design Criteria for Combined Biological Nitrogen and Phosphorus Removal

Design Parameter	Treatment Process			
	A^2/O	**Bardenpho (5 Stage)**	**UCT**	**SBR**
Food/Microorganism Ratio (kg BOD5/kg MLVSS.day)	0.15 – 0.25	0.1 – 0.2	0.1 – 0.2	0.1
Solids Retention Time (day)	4 - 27	10 - 40	10 - 30	-
MLSS (mg/L)	3000 - 5000	2000 - 4000	2000 - 4000	600 - 5000
Hydraulic Retention Time (hrs)				Batch Times
Anaerobic Zone	0.5 – 1.5	1 – 2	1 – 2	0 – 3
Anoxic Zone – 1	0.5 – 1.0	2 – 4	2 – 4	0 – 1.6
Aerobic Zone – 1	3.5 – 6.0	4 – 12	4 – 12	0.5 – 1
Anoxic Zone – 2		2 – 4	2 – 4	0 – 0.3
Aerobic Zone – 2		0.5 – 1		0 – 0.3
Settle/Decant				1.5 – 2
Total	4.5 – 8.5	9.5 - 23	9 - 22	4 – 9
Return Activated Sludge % of Influent Flowrate)	20 - 50	50 - 100	50 - 100	-
Internal Recycle (% of Influent Flowrate)	100 - 300	400	100 - 600	-

5.14.3.4. Sequencing Batch Reactor (SBR)

The SBR can be operated to achieve any combination of carbon oxidation, nitrogen reduction, and phosphorus removal. Reduction of these constituents can be accomplished with or without chemical addition by changing the operation of the reactor. Phosphorus can be removed by coagulant addition or biologically without coagulant addition. By modifying the reaction times, nitrification of nitrogen removal can also be accomplished. Overall cycle time may vary from 3 to 24 hours. A carbon source in the anoxic phase is required to support denitrification-either an external source or endogenous respiration of the existing biomass.

5.14.4. Effluent Filtration

5.14.4.1. Applicability

Effluent filtration is generally necessary when effluent quality better than 15 mg/L BOD_5, 15 mg/L suspended solids and 1.0 mg/L of phosphorus is required.

Where effluent suspended solids requirements are less than 10 mg/L, where secondary effluent quality can be expected to fluctuate significantly, or where filters follow a treatment process where significant amounts of algae will be present, a pre-treatment process such as chemical coagulation and sedimentation or other acceptable process should precede the filter units.

5.14.4.2. Design Considerations

Factors to consider when choosing between the different filtration systems which are available, include the following:

1. The installed capital and expected operating and maintenance costs;

2. The energy requirements of the systems (head requirements);

3. The media types and sizes and expected solids capacities and treatment efficiencies of the system; and

4. The backwashing systems, including type, backwash rate, backwash volume, effect on sewage works, etc.

Care should be given in the selection of pumping equipment ahead of filter units to minimize shearing of floc particles. Consideration should be given in the plant design to providing flow-equalization facilities to moderate filter influent quality and quantity.

5.14.4.3. Location of Filter System

Effluent filtration should precede the chlorine contact chamber to minimize chlorine usage, to allow more effective disinfection and to minimize the production of chloro-organic compounds.

To allow excessive biological growths and grease accumulations to be periodically removed from the filter media, a chlorine application point should be provided upstream of the filtration system (chlorine would only be dosed as necessary at this location).

5.14.4.4. Number of Units

Total filter area shall be provided in 2 or more units, and the filtration rate shall be calculated on the total available filter area with one unit out of service.

5.14.4.5. Filter Types

Filters may be of the gravity type or pressure type. Pressure filters shall be provided with ready and convenient access to the media for treatment or cleaning.

Where greases or similar solids, which result in filter plugging, are expected, filters should be of the gravity type.

5.14.4.6. Filtration Rates

5.14.4.6.1. Hydraulic Loading Rate

Filtration rates at peak hourly sewage flow rates, including backwash flows, should not exceed 2.1 L/m^{2}s for shallow bed single media systems (if raw sewage flow equalization is provided, lower peak filtration rates should be used in order to avoid under-sizing of the filter).

Filtration rates at peak hourly sewage flow rates, including backwash flows, should not exceed 3.3 L/ m^{2}s for deep bed filters (if raw sewage flow equalization is provided, lower peak filtration rates should be used in order to avoid undersizing of the filter). The manufacturer's recommended maximum filtration rate should, however, not be exceeded.

5.14.4.6.2. Organic Loading Rate

Peak solids loading rate should not exceed 50 mg/m^{2}s for shallow bed filters and 80 mg/m^{2}s for deep bed filters (if raw sewage flow equalization is provided, lower peak solids loading rates should be used in order to avoid undersizing of the filter).

5.14.4.7. Backwash

Pumps for backwashing filter units shall be sized and interconnected to provide the required rate to any filter with the largest pump out of service. Filtered water should be used as the source of backwash water. Waste filter backwash shall be adequately treated. Air scour or mechanical agitation systems to improve backwash effectiveness are recommended.

If instantaneous backwash rates represent more than 15% of the average daily design flow rate of the plant, a backwash holding tank should be provided to equalize the flow of backwash water to the plant.

5.14.4.7.1. Backwash Rate

The backwash rate shall be adequate to fluidize and expand each media layer a minimum of 20% based on the media selected. The backwash system shall be capable of providing a variable backwash rate having a maximum of at least 14 L/m^{2}s and a minimum backwash period of 10 minutes.

5.14.4.8. Filter Media

Selection of proper media size will depend on the filtration rate selected, the type of treatment provided prior to filtration, filter configuration, and effluent quality objectives. In dual or multi-media filters, media size selection must consider compatibility among media.

Table 5.27 provides minimum media depths and the normally acceptable range of media sizes. The designer has the responsibility for selection of media to meet specific conditions and treatment requirements relative to the project under consideration.

Table 5.27
Media Depths and Sizes

(Minimum Depth) (Effective Size)			
	Single Media	Multi-Media (2)	Multi-Media (3)
Anthracite	-	50 cm 1.0-2.0 mm	50 cm 1.0-2.0 mm
Sand	120 cm 1.0-4.0 mm	30 cm 0.5-1.0 mm	25 cm 06-0.8 mm
Garnet or Similar Material	-	-	5 cm 0.3-0.6 mm
Uniformity Coefficient shall be 1.7 or less			

5.14.4.9. Filter Appurtenances

The filters shall be equipped with wash water troughs, surface wash or air scouring equipment, means of measurement and positive control of the backwash rate, equipment for measuring filter head loss, positive means of shutting off flow to a filter being backwashed, and filter influent and effluent sampling points. If automatic controls are provided, there shall be a manual override for operating equipment, including each individual valve essential to the filter operation. The underdrain system shall be designed for uniform distribution of backwash water (and air, if provided) without danger of clogging from solids in the backwash water.

Provision shall be made to allow periodic chlorination of the filter influent or backwash water to control slime growths. If air is to be used for filter backwash, separate backwash blowers shall be provided.

5.14.4.10. Reliability

Each filter unit shall be designed and installed so that there is ready and convenient access to all components and the media surface for inspection and maintenance without taking other units out of service. The need for housing of filter units shall depend on expected extreme climatic conditions at the treatment plant site. As a minimum, all controls shall be enclosed. The structure housing filter controls and equipment shall be provided with adequate heating and ventilation equipment to minimize problems with excess humidity.

5.14.4.11. Backwash Surge Control

The rate of return of waste filter backwash water to treatment units should be controlled such that the rate does not exceed 15% of the design average daily flow rate to the treatment units. The hydraulic and organic load from waste backwash water shall be considered in the overall design of the treatment plant.

Surge tanks shall have a minimum capacity of two backwash volumes, although additional capacity should be considered to allow for operational flexibility. Where waste backwash water is returned for treatment by pumping, adequate pumping capacity shall be provided with the largest unit out of service.

5.14.4.12. Backwash Water Storage

Total backwash water storage capacity provided in an effluent clearwell or other unit shall equal or exceed the volume required for two complete backwash cycles.

5.14.4.13. Proprietary Equipment

Where proprietary filtration equipment not conforming to the preceding requirements is proposed, data which supports the capability of the equipment to meet effluent requirements under design conditions shall be provided. Such equipment will be reviewed on a case-by case basis at the discretion of the DOEC.

5.14.5. Microscreening

5.14.5.1. Applicability

Microscreening units may be used following a biological treatment process for the removal of residual suspended solids. Selection of this unit process should consider final effluent requirements, the preceding biological treatment process, and anticipated consistency of the biological process to provide a high quality effluent.

5.14.5.2. Design Considerations

Pilot plant testing on existing secondary effluent is encouraged. Where pilot studies so indicate, where microscreens follow trickling filters or lagoons, or where effluent suspended solids requirements are less than 10 mg/L, a pre-treatment process such as chemical coagulation and sedimentation shall be provided. Care should be taken in the selection of pumping equipment ahead of microscreens to minimize shearing of floc particles. The process design shall include flow equalization facilities to moderate microscreen influent quality and quantity.

5.14.5.3. Screen Material

The micro-fabric shall be a material demonstrated to be durable through long-term performance data. The aperture size must be selected considering required removal efficiencies, normally ranging from 20 to 35 microns. The use of pilot plant testing for aperture size selection is recommended.

5.14.5.4. Screening Rate

The screening rate shall be selected to be compatible with available pilot plant test results and selected screen aperture size, but shall not exceed 3.4 L/m^2s of effective screen area based on the maximum hydraulic flow rate applied to the units. The effective screen area shall be considered as the submerged screen surface area less the area of screen blocked by structural supports and fasteners. The screening rate shall be that applied to the units with one unit out of service.

5.14.5.5. Backwash

All waste backwash water generated by the microscreening operation shall be recycled for treatment. The backwash volume and pressure shall be adequate to assure maintenance of fabric cleanliness and flow capacity. Equipment for backwash of at least 1.65 L/m·s of screen length and 4.22 kgf/cm^2, respectively, shall be provided. Backwash water shall be supplied continuously by multiple pumps, including one standby, and should be obtained from microscreened effluent. The rate of return of waste backwash water to treatment units shall be controlled such that the rate does not exceed 15% of the design average daily flow rate to the treatment plant. The hydraulic and organic load from waste backwash water shall be considered in the overall design of the treatment plant.

Where waste backwash is returned for treatment by pumping, adequate pumping capacity shall be provided with the largest unit out of service. Provisions should be made for measuring backwash flow.

5.14.5.6. Appurtenances

Each microscreen unit shall be provided with automatic drum speed controls with provisions for manual override, a bypass weir with an alarm for use when the screen becomes blinded to prevent excessive head development, and means for dewatering the unit for inspection and maintenance. Bypassed flows must be segregated from water used for backwashing. Equipment for control of biological slime growths shall be provided. The use of chlorine should be restricted to those installations where the screen material is not subject to damage by the chlorine.

5.14.5.7. Reliability

A minimum of two microscreen units shall be provided, each unit being capable of independent operation. A supply of critical spare parts shall be provided and maintained. All units and controls shall be enclosed in a heated and ventilated structure with adequate working space to provide for ease of maintenance.

5.14.6. Activated Carbon Adsorption

5.14.6.1. Applicability

In tertiary treatment, the role of activated carbon is to remove the relatively small quantities of refractory organics, as well as inorganic compounds such as nitrogen, sulphides, and heavy metals, remaining in an otherwise well-treated wastewater.

Activated carbon may also be used to remove soluble organics following chemical-physical treatment.

5.14.6.2. Design Considerations

The usefulness and efficiency of carbon adsorption for municipal wastewater treatment depends on the quality and quantity of the delivered wastewater. To be fully effective, the carbon unit should receive an effluent of uniform quality, without surges in the flow. Other wastewater qualities of concern include suspended solids, oxygen demand, other organics such as methylene blue active substance (MBAS) or phenol, and dissolved oxygen. Environmental parameters of importance include pH and temperature. Consideration also should be given to the type of activated carbon available. Activated carbons produced from different base materials and by different activation processes will have varying adsorptive capacities. Some factors influencing adsorption at the carbon/liquid interface are:

1. Attraction of carbon for solute;
2. Attraction of carbon for solvent;
3. Solubilizing power of solvent or solute;
4. Association;
5. Ionization;
6. Effect of solvent on orientation at interface;
7. Competition for interface in presence of multiple solutes;
8. Co-adsorption;
9. Molecular size of molecules in the system;
10. Pore size distribution in carbon;
11. Surface area of carbon; and
12. Concentration of constituents.

There are several different activated carbon contactor systems that can be selected. The carbon columns can be either of the pressure or gravity type.

5.14.6.3. Unit Sizing

5.14.6.3.1. Contact Time

The contact time shall be calculated on the basis of the volume of the column occupied by the activated carbon. Generally, carbon contact times of 15 to 35 minutes are used depending on the application, the wastewater characteristics, and the desired effluent quality. For tertiary treatment applications, carbon contact times of 15 to 20 minutes should be used where the desired effluent quality is a COD of 10 to 20 mg/L, and 30 to 35 minutes when the desired effluent COD is 5 to 15 mg/L. For chemical-physical treatment plants, carbon contact times of 20 to 35 minutes should be used, with a contact time of 30 minutes being typical.

5.14.6.3.2. Hydraulic Loading Rate

Hydraulic loading rates of 2.5 to 7.0 L/m^{2}s of cross section of the bed shall be used for upflow carbon columns. For downflow carbon columns, hydraulic loading rates of 2.0 to 3.3 L/ m^{2}s are used. Actual operating pressure seldom rises above 7 kN/m^2 for each 0.3 m of bed depth.

5.14.6.3.3. Depth of Bed

The depth of bed will vary considerably, depending primarily on carbon contact time, and may be from 3 to 12 m. A minimum carbon depth of 3 m is recommended. Typical total carbon depths range from 4.5 to 6 m. Freeboard has to be added to the carbon depth to allow an expansion of 10 to 50% for the carbon bed during backwash or for expanded bed operation. Carbon particle size and water temperature will determine the required quantity of backwash water to attain the desired level of bed expansion.

5.14.6.3.4. Number of Units

A minimum of two parallel carbon contactor units are recommended for any size plant. A sufficient number of contactors should be provided to insure an adequate carbon contact time to maintain effluent quality while one column is off line during removal of spent carbon for regeneration or for maintenance.

5.14.6.4. Backwashing

The rate and frequency of backwash is dependent on hydraulic loading, the nature and concentration of suspended solids in the wastewater, the carbon particle size, and the method of contacting. Backwash frequency can be prescribed arbitrarily (each day at a specified time), or by operating criteria, (headloss or turbidity).

Duration of backwash may be 10 to 15 minutes.

The normal quantity of backwash water employed is less than 5% of the product water for a 0.8 m deep filter and 10 to 20% for a 4.5 m filter. Recommended backwash flow rates for granular carbons of 8 x 12 or 12 x 30 mesh are 8 to 14 L/m^2s.

5.14.6.5. Valve and Pipe Requirements

Upflow units shall be piped to operate either as upflow or downflow units as well as being capable of being backwashed. Downflow units shall be piped to operate as downflow and in series. Each column must be valved to be backwashed individually. Furthermore, downflow series contactors should be valved and piped so that the respective position(s) of the individual contactors can be interchanged.

5.14.6.6. Instrumentation

The individual carbon columns should be equipped with flow and headloss measuring devices.

5.14.6.7. Hydrogen Sulphide Control

Methods that can be incorporated into the plant design to cope with hydrogen sulphide production include:

1. Providing upstream biological treatment to satisfy as much of the biological oxygen demand as possible prior to carbon treatment;

2. Reducing detention time in the carbon columns based on dissolved oxygen concentrations of the effluent;

3. Backwashing the columns at more frequent intervals;

4. Chlorinating carbon column influent; and

5. In upflow expanded beds, the introducing of an oxygen source, such as air or hydrogen peroxide, to keep the columns aerobic.

5.14.6.8. Carbon Transport

Provisions must be made to remove spent carbon from the carbon contactors. It is important to obtain a uniform withdrawal of carbon over the entire horizontal surface area of the carbon bed. Care must be taken to insure that gravel or stone supporting media used in downflow contactors does not enter the carbon transport system.

Activated carbon shall be transported hydraulically. Carbon slurries can be transported using water or air pressure, centrifugal or diaphragm pumps, or eductors. The type of motive equipment selected requires a balance of owner preference, column control capabilities, capital and maintenance costs, and pumping head requirements.

Carbon slurry piping systems shall be designed to provide approximately 8.0 L of transport water for each kg of carbon removed. Pipeline velocities of 0.9 to 1.5 m/s are recommended.

Long-radius elbows or tees and crosses with cleanouts should be used at points of pipe direction change. Valves should be of the ball or plug type. No valves should be installed in the slurry piping system for the purpose of throttling flows.

5.14.6.9. Carbon Regeneration

5.14.6.9.1. Quantities of Spent Carbon

The carbon dose used to size the regeneration facilities depends on the strength of the wastewater applied to the carbon and the required effluent quality. Typical carbon dosages that might be anticipated for municipal wastewaters are shown in Table 5.28.

Table 5.28
Typical Carbon Dosages for Different Column Wastewater Influents

Pretreatment	Typical Carbon Dosage Required per m^2 of Column Throughput (g/m^3)*
Coagulated, Settled and Filtered Activated Sludge Effluent	35 – 70
Filtered Secondary Effluent	70 – 100
Coagulated, Settled, and Filtered Raw Wastewater (Physical – Chemical)	100 - 300

*Loss of carbon during each regeneration cycle typically will be 5 to 10%. Make-up carbon is based on carbon dosage and the quality of the regenerated carbon.

5.14.6.9.2. Carbon Dewatering

Dewatering of the spent carbon slurry prior to thermal regeneration may be accomplished in spent carbon drain bins. The drainage bins shall be equipped with screens to allow the transport of water to flow from the carbon. Two drain bins shall be provided.

Dewatering screws may also be used to dewater the activated carbon. A bin must be included in the system to provide a continuous supply of carbon to the screw, as well as maintain a positive seal on the furnace.

5.14.6.9.3. Regeneration Furnace
Partially dewatered carbon may be fed to the regeneration furnace with a screw conveyor equipped with a variable speed drive to control the rate of carbon feed precisely.

The theoretical furnace capacity is determined by the anticipated carbon dosage.

An allowance for furnace downtime on the order of 40% should be added to the theoretical capacity.

Based on the experience gained from two full-scale facilities, provisions should be made to add approximately 1 kg of steam per kg of carbon regenerated. Fuel requirements for the carbon regeneration furnace are 7000 kJ/kg of carbon when regenerating spent carbon on tertiary and secondary effluent applications. To this value, the energy requirements for steam and an afterburner, if required, must be added.

The furnace shall be designed to control the carbon feed rate, rabble arm speed, and hearth temperatures. The off-gases from the furnace must be within acceptable air pollution standards. Air pollution control equipment shall be designed as an integral part of the furnace and include a scrubber for removing carbon fines and an afterburner for controlling odours.

5.14.7. Constructed Wetlands
Constructed wetlands are inundated land areas with water depths typically less than 0.6 m that support the growth of emergent plants such as cattail, bulrush, reeds, and sedges. The vegetation provides surface for the attachment of bacterial films, aids in the filtration and adsorption of wastewater constituents, transfers oxygen into the water column, and controls the growth of algae by restricting the penetration of sunlight.

Although plant uptake is an important consideration in contaminant, particularly nutrient, removal it is only one of many active removal mechanisms in the wetland environment. Removal mechanisms have been classified as physical, chemical and biological and are operative in the water column, the humus and soil column beneath the growing plants, and at the interface between the water and soil columns. Because most of the biological transformations take place on or near a surface to which bacteria are attached, the presence of vegetation and humus is very important. Wetland systems are designed to provide maximum production of humus material through profuse plant growth and organic matter decomposition.

5.14.7.1. Types
Wastewater treatment systems using constructed wetlands have been categorized as either free water surface (FWS) or subsurface flow (SFS) types:

1. **Free Water Surface Wetlands (FWS) -** A FWS system consists of basins or channels with a natural or constructed subsurface barrier to minimize seepage. Emergent vegetation is grown and wastewater is treated as it flows through the vegetation and plant litter. FWS wetlands are typically long and narrow to minimize short-circuiting.

2. **Subsurface Flow Wetlands (SFS)** - A SFS wetland system consists of channels or basins that contain gravel or sand media which will support the growth of emergent vegetation. The bed of impermeable material is sloped typically between 0 and 2%. Wastewater flows horizontally through the root zone of the wetland plants about 100 to 150 mm below the gravel surface. Treated effluent is collected in an outlet channel or pipe.

5.14.7.2. Site Evaluation

Site characteristics that must be considered in wetland system design include topography, soil characteristics, existing land use, flood hazard, and climate, and are detailed as follows:

1. **Topography** - Level to slightly sloping, uniform topography is preferred for wetland sites because free water systems (FWS) are generally designed with level basins or channels, and subsurface flow systems (SFS) are normally designed and constructed with slopes of 1% or slightly more. Although basins may be constructed on steeper sloping or uneven sites, the amount of earthwork required will affect the cost of the system. Thus, slope gradients should be less than 5%.

2. **Soil** - Sites with slowly permeable (<0.5 cm/h) surface soils or subsurface layers are most desirable for wetland systems because the objective is to treat the wastewater in the water layer above the soil profile. Therefore, percolation losses through the soil profile should be minimized. As with overland-flow systems, the surface soil will tend to seal with time due to deposition of solids and growth of bacterial slimes. Compacting during construction may purposely reduce permeability of native soils. Sited with rapidly permeable soils may be used for small systems by constructing basins with clay or artificial liners. The depth of soil to groundwater should be a minimum of 0.3 - 0.6 m to allow sufficient distance for treatment of any percolate entering the groundwater.

3. **Flood Hazard** - Wetland sites should be located outside of flood plains, or protection from flooding should be provided.

4. **Existing Land Use -** Open space or agricultural lands, particularly those near existing natural wetlands, are preferred for wetland sites. Constructed wetlands can enhance existing natural wetlands by providing additional wildlife habitat and, in some cases, by providing a more consistent water supply.

5. **Climate** - The use of wetland systems in cold climates is possible. Because the principle treatment systems are biological, treatment performance is strongly temperature sensitive. Storage will be required where treatment objectives cannot be met due to low temperatures.

5.14.7.3. Preapplication Treatment

Artificial wetlands may be designed to accept wastewater with minimal (coarse screening and comminution) pretreatment. However, the level of pretreatment will influence the quality of the final effluent and therefore overall treatment objectives must be considered. Since there is no permanent escape mechanism for phosphorus within the wetland, phosphorus reduction by chemical addition is also recommended as a pretreatment step to ensure continued satisfactory phosphorus removal within the marsh.

5.14.7.4. Vegetation Selection and Management

The plants most frequently used in constructed wetlands include cattails, reeds, rushes, bulrushes, and sedges. All of these plants are ubiquitous and tolerate freezing conditions. The important characteristics of the plants related to design are the optimum depth of water for FWS systems and the depth of rhizome and root systems for SFS systems. Cattails tend to dominate in water depths over 0.15 m. Bulrushes grow well at depths of 0.05 - 0.25 m. Reeds grow along the shoreline and in water up to 1.5 m deep, but are poor competitors in shallow waters. Sedges normally occur along the shoreline and in shallower water than bulrushes. Cattail rhizomes and roots extend to a depth of approximately 0.3 m, whereas reeds extend to more than 0.6 m and bulrushes to more than 0.75 m. Reeds and bulrushes are normally selected for SFS systems because the depth of rhizome penetration allows for the use of deeper basins.

Harvesting of wetland vegetation is generally not required, especially for SFS systems. However dry grasses in FWS systems are burned off periodically to maintain free-flow conditions and to prevent channelling of the flow. Removal of the plant biomass for the purpose of nutrient removal is normally not practical.

5.14.7.5. Design Parameters

5.14.7.5.1. Detention Time

Free Water Surface Wetlands (FWS) - The relationship between BOD removal and detention times for FWS is represented by the equation:

$$Ce = Co^{(-k_T t)}$$

where: Ce = effluent BOD (mg/L);
Co = influent BOD (mg/L);
kT = temperature dependent rate constant;
$d^{-1} = k_{20} \times 1.06^{(T-20)}$;
$k_{20} = 0.678 d^{-1}$;
T = average monthly water temperature (^{o} C);
t = average detention time (d)

$$t = \frac{LWnd}{Q}$$

where: L = basin length (m);
W = basic width (m);
n = fraction of cross sectional area not used by plants (0.65-0.75);
d = depth of basin (m); and
Q = average flowrate through system [(Q_{in} + Q_{out}) / 2], (m^3/d)

$$Q = \frac{Q_{in} + Q_{out}}{2}$$

Subsurface Flow Systems (SFS) - The relationship between BOD removal and detention times for SFS is represented by the equation:

$$Ce = Co^{(-K_T t')}$$

where: Ce = effluent BOD (mg/L);
Co = influent BOD (mg/L);
K_T = temperature dependent rate constant;
$d^{-1} = K_{20} \times 1.06^{(T-20)}$
T = average monthly water temperature (^{o} C);
$K_{20} = 1.104\ d^{-1}$

$$t' = \frac{LW\alpha d}{Q}$$

where: L = basin length (m);
W = basin width (m);
α = porosity of basin medium = 0.35 (gravelly sand); 0.30 (coarse sand); 0.28 (medium sand)
d = depth of basin (m); and
Q = average flowrate through system (m^3/d)

$$Q = \frac{Q_{in} + Q_{out}}{2}$$

5.14.7.5.2. Water Depth

For FWS, the design water depth depends on the optimum depth for the selected vegetation. In cold climates, the operating depth is normally increased in the winter to allow for ice formation on the surface and to provide the increased detention time required at colder temperatures. Systems should be designed with an outlet structure that allows for varied operating depths. Water depths should range from 0.1 - 0.5 m.

The design depth of SFS systems is controlled by the depth of penetration of the plant rhizomes and roots because the plants supply oxygen to the water through the root/rhizome system. The media depth may range from 0.3 - 0.75 m.

5.14.7.5.3. Aspect Ratio

The aspect ration for FWS wetlands is important to the performance for removal of BOD, TSS, NH_3, and total nitrogen. Length to width ratios of 4:1 to 6:1 are needed to achieve expected performances and avoid short-circuiting of wastewater through the wetland. For large systems, an aspect ratio of 2:1 is the minimum recommended.

For SFS wetlands the bed width is determined by the hydraulic flowrate. The length of the bed is determined by the needed detention time for pollutant removal.

Therefore SFS wetlands may have aspect ratios less than or greater than 1:1 depending on the treatment goal.

5.14.7.5.4. Loading Rates

Table 5.29 summarizes the hydraulic, BOD, and SS loading rates for BOD removal in both FWS and SFS systems.

Table 5.29
Loading Rates for Constructed Wetlands

Wetland Type	Hydraulic Loading Rate m^3/ha.d	Maximum BOD Loading Rate kg/ha.d	Maximum SS Loading Rate at Inlet kg/m^2d
Free Water Surface	150 - 500	65	Not applicable
Subsurface Flow System	Not Applicable	65	0.08

6.14.7.5.5. Nutrient Removal

Free Water Surface Wetlands (FWS) - Detention times for nutrient removal need to be longer than the 5 - 10 days required for BOD and SS. For ammonia or total nitrogen removal, both minimum temperature and detention time are important. Detention times for significant nitrogen removal should be 8 - 14 days or more. Nitrogen removal and nitrification will be reduced when water temperatures fall below 10ºC and should not be expected when water temperatures fall below 4ºC.

Plant uptake of phosphorus is rapid, and following plant death, phosphorus may be quickly recycled to the water column or deposited in the sediments. The only major sink for phosphorus in most wetlands is in the soil. Significant phosphorus removal requires long detention times (15 - 20 days) and low phosphorus loading rates (< 0.3 kg/ha·d).

Subsurface Flow Wetlands (SFS) - Both detention time and oxygen transfer can limit nitrification and subsequent nitrogen removal in SFS wetlands. Because nitrification of 20 mg/L of ammonia will require 100 mg/L of oxygen, oxygen transfer is critical to nitrification in SFS wetlands. Plant roots can generate a portion of this demand for oxygen in the subsurface, however, direct oxygen transfer from the atmosphere may be required to achieve effective nitrification. The detention time and temperature limits for FWS apply to SFS wetlands.

5.14.7.6. Vector Control

FWS systems provide ideal breeding habitat for mosquitoes. Plans for biological control of mosquitoes through the use of mosquito fish and sparrows plus application of chemical control agents as necessary must be incorporated in the design. Thinning of vegetation may also be necessary to eliminate pockets of water that are inaccessible to fish.

Mosquito breeding should not be a problem in SFS systems, provided the system is designed to prevent mosquito access to the subsurface water zone. The surface is normally covered with pea gravel or coarse sand to achieve this purpose.

5.14.7.7. Vegetation Harvesting

Harvesting of the emergent vegetation is only required to maintain hydraulic capacity, promote active growth, and avoid mosquito growth. Harvesting for nutrient removal is not practical and is not recommended.

5.14.7.8. Monitoring

Monitoring is necessary to maintain loadings within design limits. A routine monitoring program should be established for the following parameters:

1. Wastewater application rates (m^3/d);
2. Discharge flow rates (m^3/d);
3. Wastewater quality, including BOD_5 and COD, suspended solids, total dissolved solids, total nitrogen, total phosphorous, pH and sodium adsorption ratio; and
4. Discharge water quality according to the analyses summarized in item 3 above.

5.14.8. Floating Aquatic Plant Treatment Systems

Aquatic treatment systems consist of one or more shallow ponds in which one or more species of water tolerant vascular plants such as water hyacinths or duckweed are grown. The shallower depths and the presence of aquatic macrophytes in the place of algae are the major differences between aquatic treatment systems and stabilization ponds. The presence of plants is of great practical significance because the effluent from aquatic systems is of higher quality than the effluent from stabilization pond systems for equivalent or shorter detention times. This is true, particularly when the systems are situated after conventional pond systems, which provide greater than primary treatment.

In aquatic systems, wastewater is treated principally by bacterial metabolism and physical sedimentation, as is the case in conventional trickling filter systems. The aquatic plants themselves bring about very little actual treatment of the wastewater. Their function is to provide components of the aquatic environment that improve the wastewater treatment capability and/or reliability of that environment.

5.14.8.1. Plant Selection

The principal floating aquatic plants used in aquatic treatment systems are water hyacinth, duckweed and pennywort. These plants are described in greater detail in the following discussion.

5.14.8.1.1. Water Hyacinths

Water hyacinth is a perennial, fresh water aquatic vascular plant with rounded, upright, shiny green leaves and spikes of lavender flowers. The petioles of the plant are spongy with many air spaces and contribute to the buoyancy of the hyacinth plant. When grown in wastewater, individual plants range from 0.5 to 1.2 m from the top of the flower to the root tips. The plants spread laterally until the water surface is covered, and then the vertical growth increases. The growth of water hyacinth is influenced by efficiency of the plant to use solar energy, nutrient composition of the water, cultural methods, and environmental factors.

Under normal conditions, loosely packed water hyacinths can cover the water surface at relatively low plant densities, about 10 kg/m^2 wet weight. Plant densities as high as 80 kg/m^2 wet weight can be reached. As in other biological processes, the growth rate of water hyacinths is dependent on temperature. Both air and water temperatures are important in assessing plant vitality.

5.14.8.1.2. Duckweed

Duckweed is a small, green freshwater plant with fronds from one to a few millimetres in width with a short root, usually less than 12 mm in length. Duckweed is the smallest and the simplest of the flowering plants and has one of the fastest reproduction rates. Duckweed grown in wastewater effluent, at 27ºC, doubles in frond numbers, and therefore in area covered, every four days. The plant is made up of essentially metabolically active cells with very little structural fibre.

Small floating plants, particularly duckweed, are sensitive to wind and may be blown in drifts to the leeward side of the pond unless baffles are used. Redistribution of the plants requires manual labour. If drifts are not redistributed, decreased treatment efficiency may result due to incomplete coverage of the pond surface. Odours have also developed where accumulated plants are allowed to remain and undergo anaerobic decomposition.

5.14.8.1.3. Pennywort

Pennywort is generally a rooted plant. However, under high-nutrient conditions, it may form hydroponic rafts that extend across water bodies. Pennywort tends to intertwine and grows horizontally; at high densities, the plants tend to grow vertically. Unlike water hyacinth, the photosynthetic leaf area of pennywort is small, and, at dense plant stands, yields are significantly reduced as a result of self-shading. Pennywort exhibits mean growth rates greater than 0.010 kg/m^2d in warm climates. Although rates of nitrogen and phosphorous uptake by water hyacinth drop sharply during the winter, nutrient uptake by pennywort is approximately the same during both warm and cool seasons. Pennywort is a cool season plant that can be integrated into water hyacinth/water lettuce biomass production systems.

5.14.8.2. Types of Systems

The principal types of floating aquatic plant treatment systems used for wastewater treatment are those employing water hyacinth and duckweed.

5.14.8.2.1. Water Hyacinth Systems

Water hyacinth systems represent the majority of aquatic plant systems that have been constructed. Three types of hyacinth systems can be described based on the level of dissolved oxygen and the method of aerating the pond:

1. Aerobic non-aerated;
2. Aerobic aerated; and
3. Facultative anaerobic.

A non-aerated aerobic hyacinth system will produce secondary treatment or nutrient (nitrogen) removal depending on the organic-loading rate. This type of system is the most common of the hyacinth systems now in use. The advantages of this type of system include excellent performance with few mosquitoes or odours.

For plant locations in which no mosquitoes or odours can be tolerated, an aerated aerobic hyacinth system is required. The added advantages of such a system are that with aeration, higher organic-loading rates are possible, and reduced land area is required.

The third configuration for a hyacinth system is known as a facultative anaerobic hyacinth system. These systems are operated at very high organic-loading rates.

Odours and increased mosquito populations are the principal disadvantages of this type of system. Facultative anaerobic hyacinth systems are seldom used because of these problems.

5.14.8.2.2. Duckweed Systems

Duckweed and pennywort have been used primarily to improve the effluent quality from facultative lagoons or stabilization ponds by reducing the algae concentration. Conventional lagoon design may be followed for this application, except for the need to control the effects of wind. Without controls, duckweed will be blown to the downwind side of the pond, resulting in exposure of large surface areas and defeating the purpose of the duckweed cover. As noted previously, accumulations of decomposing plants can also result in the production of odours.

Floating baffles can be used to construct cells of limited size to minimize the amount of open surface area exposed to wind action.

5.14.8.3. Climatic Constraints

The water hyacinth systems that are currently used to treat wastewater are located in the warm temperature climates. The optimum water temperature for water hyacinth growth is 21 – 30EC. Air temperatures of –3EC for 12 hours will destroy the leaves and exposure at –5EC for 48 hours will kill the plants. If a water hyacinth system were to be used in a colder climate, it would be necessary to house the system in a greenhouse and maintain the temperature in the optimum range. Duckweed is more cold tolerant than water hyacinths and can be grown practically at temperatures as low as 7EC.

5.14.8.4. Preapplication Treatment

The minimum level of pre-application treatment should be primary treatment, short detention time aerated ponds or the equivalent. Treatment beyond primary depends on the effluent requirements. Use of oxidation ponds or lagoons in which high concentrations of algae are generated should be avoided prior to aquatic treatment because algae removal is inconsistent. When there are effluent limitations on phosphorus, it should be removed in the pre-application treatment step because phosphorus removal in aquatic treatment systems is minimal.

5.14.8.5. Design Parameters

The principal design parameters for aquatic treatment systems include hydraulic detention time, water depth, pond geometry, organic-loading rate, and hydraulic loading rate. Typical design guidelines for water hyacinth and duckweed systems are summarized in Table 5.30 for different levels of pre-application treatment.

Table 5.30
Floating Aquatic Plant System Design Criteria

Item	**Type of Water Hyacinth Treatment System**			**Duckweed Treatment System**
	Secondary Aerobic (non-aerated)	**Secondary Aerobic (aerated)**	**Nutrient Removal Aerobic (non-aerated)**	
Influent Wastewater	Primary Effluent	Primary Effluent	Secondary Effluent	Facultative Pond Effluent
Influent BOD_5 (mg/L)	130 - 180	130 - 180	30	40
BOD_5 Loading (kg/ha.d)	45 - 90	170 - 340	10 - 45	22 – 28
Water Depth (m)	0.5 – 1.0	1.0 – 1.3	0.7 – 1.0	1.3 – 2.0
Detention Time (d)	10 - 36	4 - 8	6 - 18	20 – 25
Hydraulic Loading Rate (m^3/ha.d)	190 - 570	95 - 285	375 - 1500	570 – 860
Water Temperature (ºC)	> 10	> 10	> 10	> 7
Harvest Schedule	Seasonally	Bi-monthly	Bi-monthly	Monthly

5.14.8.6. Pond Configuration

5.14.8.6.1. Water Hyacinth Systems

Typical pond configurations used for water hyacinth systems involve rectangular basins in series similar to stabilization ponds. Recycle and step feed are employed to reduce the concentration of the organic constituent at the plant root zone, improve the transport of wastewater to the root zone, and reduce the formation of odours.

5.14.8.6.2. Duckweed Systems
Duckweed systems should be designed as conventional stabilization ponds except for the need to control the effects of wind. Floating baffles are used to minimize the amount of surface area exposed to direct wind action. Without this control, duckweed will be blown by the wind and treatment efficiencies cannot be achieved.

5.14.8.7. Plant Harvesting and Processing
The need for plant harvesting depends on water quality objectives, the growth rates of the plants, and the effects of predators such as weevils. Harvesting of aquatic plants is needed to maintain a crop with high metabolic uptake of nutrients. Frequent harvesting of hyacinths is practiced to achieve nutrient removal. Significant phosphorus removal is achieved only with frequent harvesting. In areas where weevils pose a threat to healthy hyacinth populations, selective harvesting is often used to keep the plants from being infected. Duckweed harvesting for nutrient removal may be required as often as once per week during warm periods.

Harvested water hyacinth plants are typically dried and landfilled or spread on land and tilled into the soil. Water hyacinth can also be composted readily. However, if the plants are not first partially dried or squeezed, the high moisture content tends to reduce the effectiveness of the compost process and results in the production of a liquid stream that must be disposed of. Ground duckweed can be used as animal feed without air-drying.

5.15. Septic Tank Systems

Septic tanks are principally used for the treatment of household wastes from individual residences. However, if the land availability, soil conditions and other specific criteria are acceptable larger installations can be considered.

Any septic tank system that is part of a municipal system is the responsibility of the DOEC, and all non-municipal systems fall under the jurisdiction of the Department of Government Services.

5.15.1. Definitions
Septic Tank - A water tight, covered receptacle designed and constructed to receive the discharge of sewage from a building sewer; separate solids from the liquid; digest organic matter and store digested solids through a period of detention and allow the clarified liquids to discharge for disposal.

Tile Bed - A system of absorption trenches laid in parallel to distribute the septic tank effluent.

Sand Filter - A bed consisting of a number of lines of perforated pipe or drain tile surrounded by clean, coarse aggregate, containing an intermediate layer of sand as filtering material and provided with a system of underdrains for carrying off the filtered septic tank effluent.

5.15.2. Septic Tank

5.15.2.1. Location

The following are minimum clearance distances, however local conditions may necessitate greater distances:

1. 15 m from any well, lake, stream or pond;
2. 7.5 m from water service line;
3. 15 m from any watermain;
4. 1.5 m from any building; and
5. 3 m from any property boundary.

5.15.2.2. Capacity

The tank shall not be designed to receive weeping tile or roof drainage. Larger tanks are more economical with regard to maintenance, since they do not have to be cleaned as often.

5.15.2.3. Tank Proportions

The length should not be less than 2 or more than 4 times the width.

A minimum of two compartments is required. The volume of the first compartment is to be approximately equal to two-thirds of the total volume.

A typical detail of a septic tank is detailed as Figure 5.3.

5.15.2.4. Detention Time

A primary treatment unit shall have a minimum detention time of 24 hours using average daily flows.

5.15.2.5. Design Flow

When insufficient data is available the values listed in Table 5.1 (page 5-4) may be used in computing thr design flow.

5.15.2.6. Liquid Depth

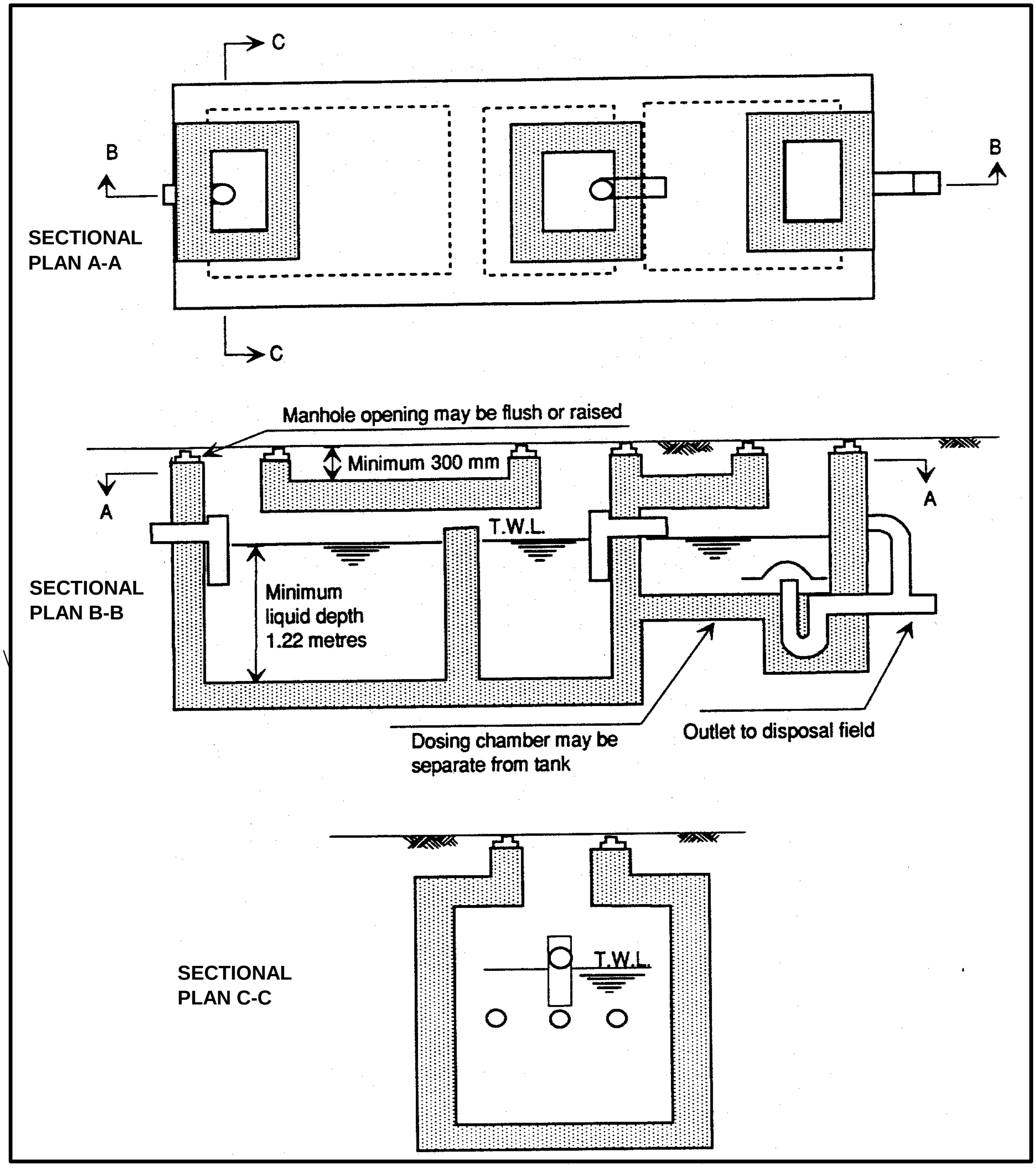
A minimum liquid depth of l.22 m is to be provided.

5.15.2.7. Inlets and Outlets

Centred tee-type inlet and outlet are acceptable. Alternatively baffled weir type outlets can be provided. The invert of the outlet shall be a minimum of 75 mm below the invert of the inlet.

Every tank must have vents in order to prevent sewage flows from draining by vacuum “u”-traps in house plumbing and to also allow the escape of methane and malodorous gases from the tank. In this regard, inlet and outlet tees shall not be capped.

Figure 5.3
Typical Septic Tank Detail



5.15.2.8. Top of Tank
The top of the tank shall be at least 300 mm below the finished grade. There should be at least 25 mm clearance between the underside of the top of the tank and the top of partitions or baffles.

5.15.2.9. Tank Installation
Pre-cast tanks shall be installed according to manufacturer's requirements. Precast/Cast In Place Tanks should be placed on properly designed Type 2 granular bedding in accordance with the Department of Municipal and Provincial Affairs Water, Sewer and Roads Master Specification Section 02223.

5.15.2.10. Manholes
At least one manhole with a minimum dimension of 600 mm shall be provided in each compartment. The manhole is to extend to the ground surface to facilitate inspection and sludge and scum removal. The manholes should be located above the inlet and outlet points.

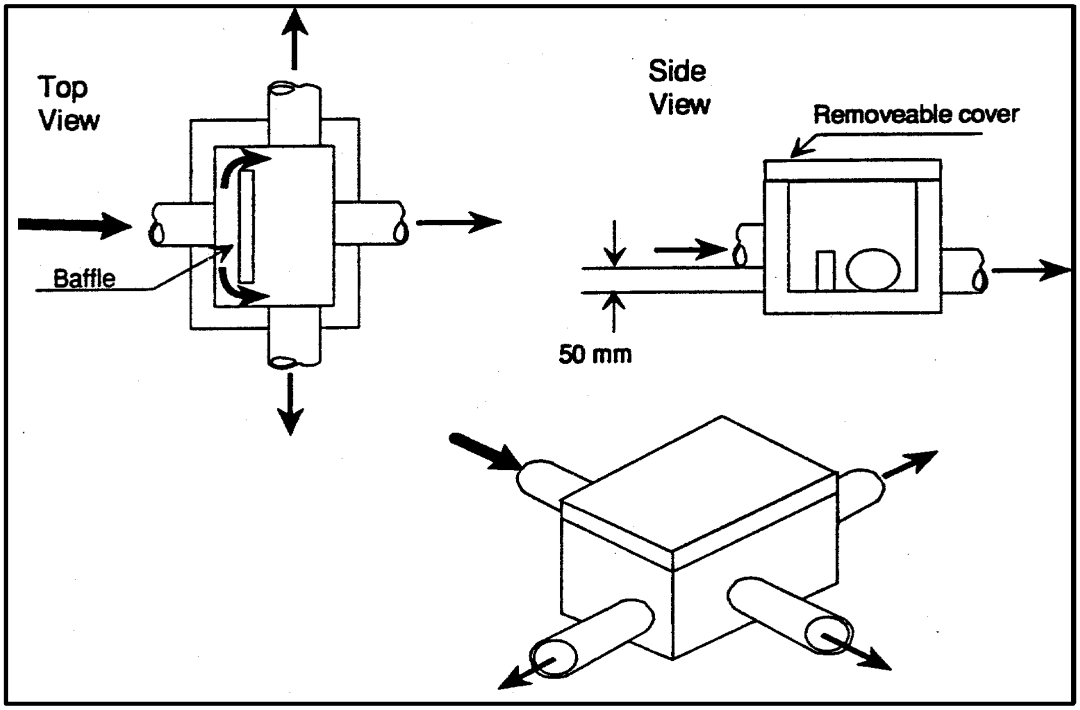
5.15.2.11. Material of Construction
The tank shall be watertight and constructed of durable material not subject to corrosion, decay, and frost damage or cracking. The walls of poured in place concrete tanks shall be at least 150 mm thick and adequately reinforced.

5.15.2.12. Dosing Devices
If a distribution field is being used and is has a length greater than 150 m, the tank shall have a pump or siphon designed to discharge a volume of sewage equal to approximately 70% of the volume of the distribution field. A siphon or pumped dosing system may be used depending on the circumstances. The dosing chamber shall have sufficient volume to provide the desired dosing volume, plus a reserve volume if it is a pumped system. In the case of a pumped system, the reserve volume is the volume of the chamber between the high water alarm switch and the invert of the inlet pipe. Also, in the case of a pumped system, an alarm system must be provided to indicate that a problem has occurred.

5.15.3. Distribution Boxes
Adequate frost protection must be provided. The distribution box shall be installed level to ensure that there is an equal splitting of flows. See Figure 5.4 for distribution box details.

Note: there have been some operational problems with distribution boxes in Newfoundland and Labrador.

Figure 5.4
Distribution Box Detail



5.15.4. Tile Field

5.15.4.1. Location

The following are minimum clearance distances:

1. 30 m to the nearest dug well or other source of water supply;
2. 15 m to a drilled well, which has a casing to at least 7.5 m below ground;
3. 15 m to a building;
6. 7.5 m from water service line;
7. 15 m from any watermain;
8. 3 m to a property boundary; and
9. 30 m to any lake, stream or pond.

5.15.4.2. Design

1. For undisturbed soil only, a tile field serving a building or structure other than a private dwelling having four bedrooms or less shall have a total length in metres of distribution pipe which shall be greater than or equal to the value determined from the following formula:

$$L = 0.011Q\sqrt{T}$$

Where: L = total length of distribution pipe (m);
Q = total daily sewage flow (L); and
T = percolation time (min).

2. For imported fill percolation tests see Table 5.31;

3. The minimum percolation time for imported fill shall be 5 minutes;

4. The bottom of a tile field shall be at all points at least 0.9 m above the maximum elevation of the groundwater table or rock or other impervious stratum located in the area of the bed.

5. Absorption trenches shall be at least 450 mm in width and shall be between 600 mm and 900 mm in depth.

6. Distribution pipe shall be located l.8 m or more apart.

7. Distribution pipe shall have a diameter not less than 100 mm and a uniform slope of between 0.33% and 0.5%, except if a siphon or pump is to be used when a uniform slope of between 0.25% and 0.33% shall be used.

8. Distribution pipe shall be surrounded by gravel or broken stone 19 mm to 28 mm in size from a level 150 mm below the bottom of the pipe to a level 75 mm above the pipe.

9. Distribution piping shall have the ends interconnected or capped.

10. Filter fabric shall be placed over the stone or broken gravel to prevent the migration of silt into the absorption trenches.

11. See Figure 5.5.

Table 5.31
Commonly Used Fill Materials and their Design Infiltration Rates

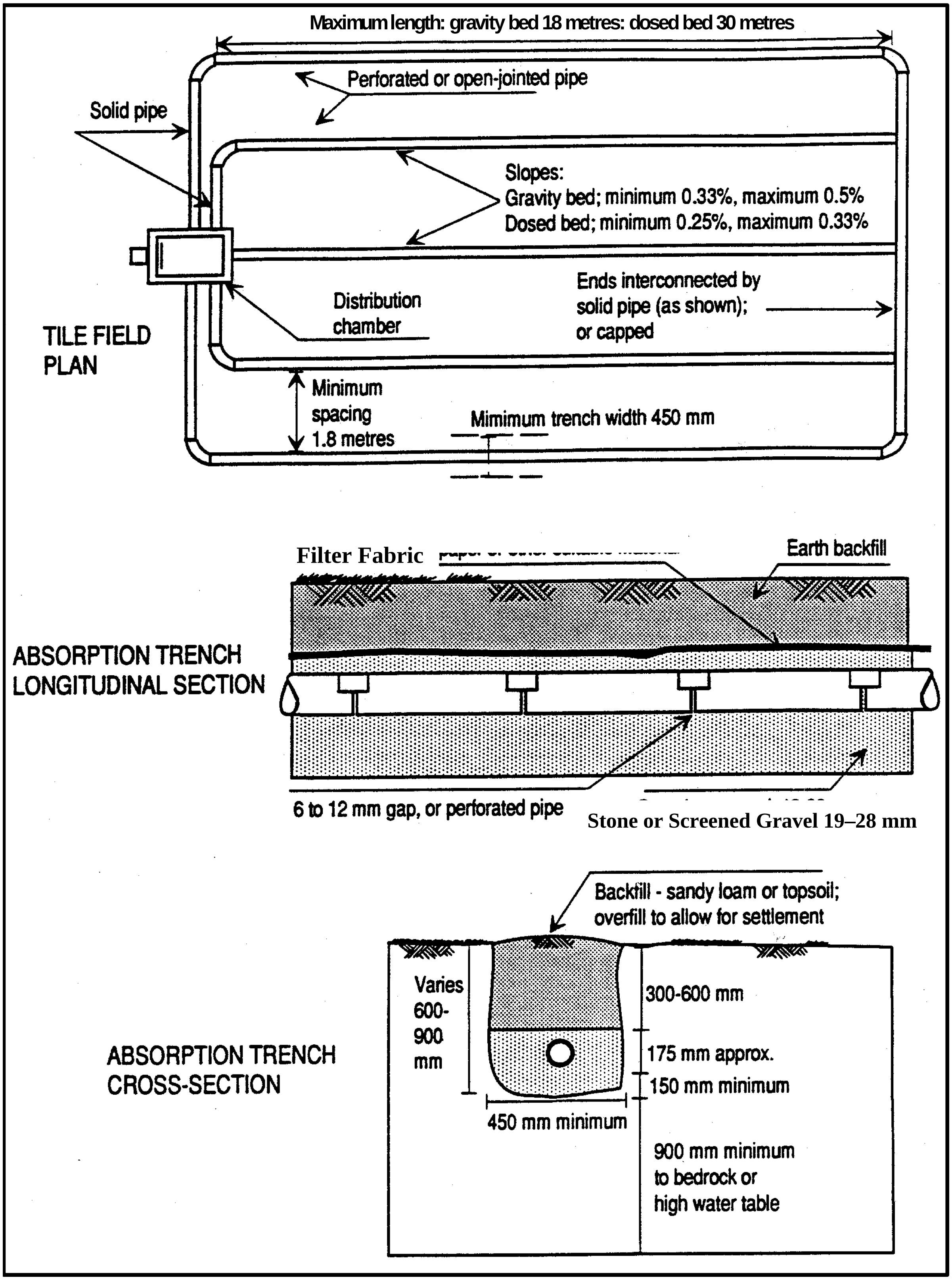
Fill Material	Characteristics[1]	Design Infiltration Rate (L/d/m^2)
Medium Sand	> 25% 0.25 – 2.0 mm < 30 - 35% 0.05 – 0.25 mm < 5 - 10% 0.002 – 0.05 mm	49
Sandy Loam	5 – 15% Clay Content	25
Sand/Sandy Loam Mixture	88 – 93 % Sand 7 – 12% Finer Grained Material	49
Bottom Ash	-	49

[1] Percent by weight

5.15.4.3. Construction

Grading of the site should be completed before the system is installed. The interface between the existing ground and the absorption field shall be scarified. After installation, heavy equipment and vehicular traffic shall be excluded from the tile field area.

Figure 5.5
Tile Field Details



5.15.4.4. Site Appraisal and Soil Percolation Tests

The most important step for subsurface disposal is the appraisal of soil and site conditions. Test pits should be dug throughout the site to ensure a complete knowledge of the various factors affecting the design. Observations should be made of soil type and conditions, the distance to groundwater, the distance to bedrock or other impervious stratum and any other factors, which influence the design.

The suitability of the soil for absorbing the liquid shall be estimated by soil percolation tests.

A percolation test shall be conducted as follows:

1. An excavation shall be made in the soil at the site where the leaching bed is to be located;

2. The excavation shall have the following dimensions:
   a) The diameter shall be between 100 mm and 300 mm; and
   b) The depth shall be the distance between the ground level and the bottom of the proposed leaching bed.
3. All loose material and smeared clay shall be removed from the sides and the bottom of the excavation;

4. The bottom of the excavation shall be covered with 50 mm of sand or fine gravel;

5. Clear water shall be poured into the excavation to a depth of at least 300 mm;

6. Additional clear water shall be added as may be necessary to maintain a depth of water of at least 300 mm in the excavation until the soil in the area of the excavation has become swollen and saturated with water and the water being added to the excavation seeps away at a constant rate; and

7. The average time in minutes required for the water to drop 25 mm shall be determined.

8. Refer to Table 5.32.

9. See Figure 5.6.

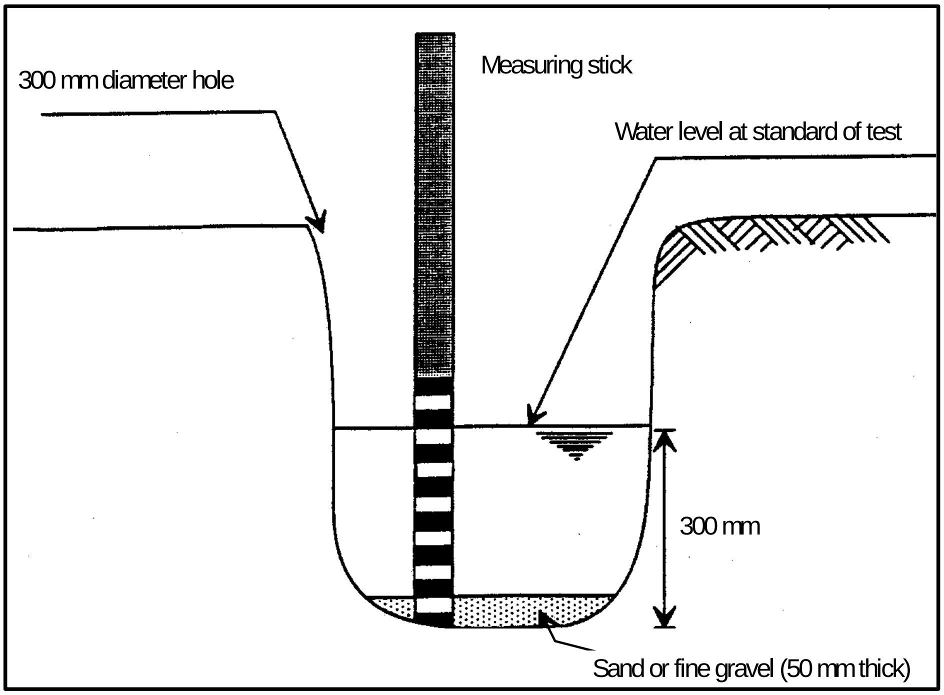
Table 5.32
Required Absorptive Area

Time for 25 mm fall (minutes)	**Application Rate (m^3/m^2d)**
0 – 5	0.117
6 – 7	0.103
8 – 10	0.083
11 – 15	0.064
16 – 20	0.054
21 – 30	0.039
Over 30	Questionable Suitability

5.15.5. Other Distribution Methods

In soil that is relatively impermeable tile field trenches are not satisfactory. They become more expensive as the soil permeability decreases and may not be feasible when the percolation time exceeds 30 or 40 min. Therefore when absorption trenches are impractical, the possibility of treating septic tank effluent using other disposal methods such as sand filter trenches, mound system, contour trench, peat beds, constructed wetlands, etc. should be considered.

Figure 5.6
Percolation Test Arrangement



5.16. Holding Tanks

5.16.1. Policy

Before a permit is issued for a holding tank, the owner should have an agreement with a licensed contractor for the disposal of the holding tank waste.

5.16.2. Tank Construction and Installation

The following are guidelines for the construction of any tank used in a sewage system for collecting, holding or storing sewage:

1. The tank shall be constructed of concrete, steel, fibreglass, reinforced plastic, or such other material, as may be approved.

2. The tank installed, assembled or constructed shall:
   a) Be carefully made, exhibit craftsmanship and true quality, be sound, durable and thoroughly capable of satisfactory and trouble-free service;
   b) Have such wall thickness, reinforcing and strength as is necessary to meet the requirements of use in service and any stresses to which it may be subjected prior to or during installation; and
   c) Be of watertight construction.

3. Materials used in the construction of the tank shall meet the appropriate industry standards and codes applicable to such materials.

4. The tank shall be constructed or installed on site according to good construction practice or, where specified, the manufacturer's recommendations, and in a manner that will ensure against:
   a) Subsequent settlement of the tank or subsequent uplift due to ground water pressure, that will be disruptive to the sewage system of which the tank is a part;
   b) Damage to the tank or any protective coating during handling and backfilling; and
   c) Damage to the tank due to weather or frost.

5. A prefabricated holding tank shall be constructed so as to meet the requirements for certification by:
   a) The Canadian Standards Association;
   b) The Underwriters Laboratories of Canada; or
   c) An organization accredited by the Standards Council of Canada for certifying products of a type that include such tanks.

6. A tank constructed on-site of concrete shall be poured-in-place and shall:
   a) Comply with the requirements of Canadian Standards Association Standard A23.1;
   b) Have a balanced design of steel reinforcing sufficient to meet the requirements of Clause b;
   c) Have bar or wire reinforcement in accordance with the requirements of Canadian Standards Association Standard A23.3;
   d) Conform to the general requirements appearing in clause 3 of Canadian Standards Association Standard B66 except as otherwise provided herein;
   e) Have top and bottom slabs of a thickness of at least 10 cm in the case of a tank having a capacity of 9000 L and at least 15 cm for a tank having a capacity in excess of 9000 L;
   f) Have sides and ends with a thickness of at least 15 cm in the case of a tank having a capacity of 27 000 L or less and at least 20 cm for a tank having a capacity in excess of 27 000 L; and
   g) have an inlet and outlet of such size as required to accommodate the sewage flows.

The following are guidelines for the construction of a holding tank, which is to be used for the holding or storage of sewage, prior to its collection:

1. The holding tank shall have an apparatus or device installed and kept operating to provide a warning, which is visual or audible or both to indicate when the tank is nearing capacity and should be emptied;

2. The holding tank shall be of a design and construction that will allow it to be sealed in such a manner as to be capable of withstanding internal pressure as specified in Underwriters Laboratories of Canada Standard;

3. The holding tank shall be of a design and construction as will allow the complete removal of solid matter that can be expected to settle in the holding tank;

4. The holding tank shall have an apparatus or device suitable for allowing the contents of the holding tank to drain from or be otherwise removed from the holding tank and any such apparatus or device shall have a suitable lock-out device;

5. The lock-out device shall remain locked or sealed except when it is necessary to unlock or unseal the device for the purpose of repairing, servicing, cleaning or emptying the holding tank and the lock-out device shall not be unlocked or unsealed except by a person engaged in the business of repairing, servicing, cleaning or emptying holding tanks or by a provincial officer;

6. The working capacity of a holding tank shall not be less than 9000 L; and

7. The holding tank shall have an apparatus or device capable of preventing the overflow of sewage from the holding tank.

5.17. Communal Septic Tank Systems

Communal Septic Tank Systems are generally used for the collection and primary treatment of sewage waste from a municipal sewer system. Approval of municipally owned and operated communal septic tank systems, regardless of capacity, is the responsibility of the DOEC, as per Section 36 of *The Water Resources Act,* SNL 2002 cW-4.01.

5.17.1. Location

The following are minimum clearance distances, however local conditions may necessitate greater distance:

1. 15 m from any well, lake, stream or pond;
2. 1.5 m from any building;
3. 3 m from any property boundary;
4. 7.5 m from a water service line; and
5. 15 m from a water distribution main.

5.17.2. Capacity

Tanks will be sized to provide a minimum of 24 hours retention at the design flow capacity to provide solids settling time. Tank liquid capacity will be based on the following criteria:

1. Actual measured flows; **OR**
2. 340 L/capita/day, if using number of persons; **OR**
3. Number of houses x 3.0 x 340 litres, if using number of homes to be connected; **PLUS**
4. Any additional extraneous or other flows, on case-by-case basis.

Examples: 100 persons: 340 L x 100 = 34 000 L (34 m^3)

30 homes: 30 x 3.0 x 340 L = 30 600 L (30 m^3)

In addition to liquid capacity, the tank must provide a minimum of 300 mm storage between the water level and the top of the tank to allow for scum storage.

The minimum liquid depth in the tank shall be 1220 mm.

5.17.3. Solids Retention - Inlets and Outlets

This section specifies requirements for the inlet/outlet device of the communal septic tank and is in addition to section 3.2.3 of CAN/CSA-B66-M90. Inlets and outlets shall be of the centred tee type to permit rodding when necessary. Inlet and outlet pipes shall extend below the liquid level by 360 mm, and extend above the liquid depth by 200 mm.

5.17.4. Maintenance Requirements

While communal septic tanks do not require daily operator attention, regular, periodic maintenance is required to ensure efficient operation. Septic tanks receiving domestic sewage accumulate sludge at a rate of roughly 65 L/capita/year. Maintenance objectives include:

1. Prevention of tank blockages that result in sewer system backups;
2. Prevention of odour problems resulting in complaints;
3. Periodic sludge and scum level measurements to predict removal needs;
4. Prevention of solids discharge to downstream small diameter sewers, disposal beds, and receiving waters; and
5. Removal of sludge and scum when necessary.

5.17.5. Material of Construction

The tank shall be made of sound and durable material. Any fitting, pipe, baffle, device partition, or other component part shall be compatible with the tank. Prefabricated communal septic tanks shall conform to CAN/CSA-B66-M90 or the latest edition in effect at the time of construction. Cast in place concrete communal septic tanks shall conform to CAN3-A23.1-M90 and testing in accordance with CAN3-A23.2-M90 or the latest edition in effect at the time of construction. Cast in place concrete tanks shall have a minimum wall thickness of 150 mm and be properly reinforced.

5.17.6. Top of Tank

The top of the tank shall be at least 300 mm below the finished grade. There should be at least 300 mm clearance between the underside of the top of the tank and the liquid level of the tank.

5.17.7. Bedding

Pre-cast tanks shall be installed according to manufacturer's requirements. Precast/Cast In Place Tanks should be placed on properly designed granular bedding type 2 in accordance with the Department of Municipal and Provincial Affairs Water, Sewer and Roads Master Specification Section 02223.

5.17.8. Access

At least one manhole with a minimum dimension of 600 mm shall be provided in each compartment. The manhole is to extend to the ground surface to facilitate inspection and sludge and scum removal. The manholes shall be located above the inlet and outlet points.

5.17.9. Scum Capacity

Scum capacity for the communal septic tank shall conform to CAN/CSA -B66-M90 Section 3.2.4 or the latest edition in effect at the time of construction.

5.17.10. Effluent Disposal Options

5.17.10.1. Ocean/Marine Discharge

Tank effluent may be discharged to an ocean or marine receiving waters. Outfall location and design will be based on the assimilative capacity of the water. Refer to Section 5.2.15.

5.17.10.2. Exfiltration Beds

Tank effluent may be discharged to an exfiltration bed/leach chamber installed between high tide mark and low tide mark where conditions permit.

5.17.10.3. Communal Disposal Beds

Tank effluent may be discharged to large conventional disposal beds. Reference shall be made to Section 5.15.4.

5.17.10.4. Distribution System

Tank effluent may also be discharged into a small diameter distribution system (SDDS) (see Section 5.2.14.7). The SDDS then transports the effluent to a properly designed area such as a marine outfall, treatment facility etc., or to a conventional municipal sewage collector system. The transport of effluent through the SDDS can normally be achieved through gravity, however on site conditions may necessitate pumping.

5.17.10.5. Other Innovative Means of Disposal

It is recognized that technological advances may present alternative options for disposal of septic tank effluents. The DOEC may review and consider such options. Acceptability will generally be based on site-specific requirements, and will usually require proof of acceptability in other jurisdictions.

5.18. Travel Trailer Dumping Stations

The design of travel trailer dumping stations must take into consideration and make provision for the possibility of sewage being spilled during the dumping process. This should be addressed by installing a sloped concrete pad and adding a potable water source for wash down and hygienic purposes with appropriate provisions to avoid cross contamination with the water source through potential backflow situations.

The use of a sloped concrete pad design will permit spilled sewage and wash down water to be directed to a ground level drain equipped with a hinged cap that is self-closing and vandal proof. When using this design, an approved backflow prevention device must be permanently installed on the potable waterline. The wash down hose should also be permanently attached to a wooden or steel post in order to restrict its movement to the immediate area of the dumping station. To prevent parking lot drainage from entering the dumping station, the concrete pad for the dumping station should be raised above the adjoining parking lot or roadway.

5.19. Disinfection of Sewerage

Disinfection of sewerage treatment plant effluent shall be required in all cases, except where otherwise approved by the DOEC.

The design shall consider meeting both the bacterial standards and the disinfectant residual limit in the effluent. The disinfection process should be selected after due consideration of waste characteristics, type of treatment process provided prior to disinfection, waste flow rates, pH of waste, disinfectant demand rates, current technology application, cost of equipment and chemicals, power cost, and maintenance requirements.

5.19.1. Forms of Disinfection

Chlorine is the most commonly used chemical for wastewater effluent disinfection. The forms most often used are liquid chlorine and calcium or sodium hypochlorite. Other disinfectants, including chlorine dioxide, ozone, bromine or UV may be accepted by the DOEC in individual cases. If chlorination is utilized, it may be necessary to dechlorinate if the chlorine level in the effluent would impair the natural aquatic habitat of the receiving body of water. The use of chlorine capsules may be considered for small systems.

5.19.2. Chlorination

5.19.2.1. Design Guidelines

5.19.2.1.1. Mixing

The disinfectant shall be mixed as rapidly as possible, with a complete mix being completed in 3 seconds. This may be accomplished by either the use of a turbulent flow regime or a mechanical flash mixer.

5.19.2.1.2. Diffusers

A chlorine solution diffuser shall be placed ahead of the contact tank and near the vicinity of the mixing area.

5.19.2.1.3. Contact Time and Residual

A total chlorine residual of 0.5 mg/L is generally required. The required detention time shall be based upon the more stringent of either 30 minutes at design average daily flow or 15 minutes at the design peak hourly flow. The criteria to be used in the design shall be that which provides the largest volume for the contact tank.

5.19.2.1.4. Coliform Levels

Acceptable effluent coliform levels shall be based upon the results of the receiving water study and the receiving water quality guidelines.

5.19.2.1.5. Contact Tank

In order that the chlorine contact tank can provide the required detention, dead zones within the tank must be avoided and the flow through the tank must approach plug flow as closely as possible. Back mixing within the contact tank must be avoided to prevent short-circuiting and the resulting poor disinfection results. Covered tanks are discouraged.

To approach a plug-flow regime, flow channels with length-to-width ratios of greater than 40:1 are required. Length-to-width ratios of 10:1 produce detention times of approximately 70 % of the theoretical residence times. In rectangular tanks, longitudinal baffling to produce long, narrow flow channels with a serpentine flow pattern and with guide vanes at changes in direction is a preferred method.

Since some sedimentation occurs in chlorine contact tanks, provision should be made for periodic sludge removal from the chlorine contact tank(s). The drain should be valved. If it is necessary to take the contact tank out of operation for cleaning, and if short-term discontinuation of disinfection cannot be tolerated due to other critical uses made of the receiving waters, two contact basins shall be provided. In less critical situations, one contact basin will suffice provided that the bypass facilities are equipped with a chlorine application point for emergency disinfection.

5.19.2.2. Chlorination Facilities Design

Refer to Sections 4.2.4 and 4.2.5

5.19.2.2.1. Feed Equipment

For normal domestic sewage, Table 5.33 may be used as a guide in sizing chlorination facilities.

In order that effective disinfection can be maintained at all times, without the need to overdose excessively at low flow periods, the chlorine feed equipment should be paced by the effluent flow rate.

For additional information regarding feed equipment, including capacity, standby equipment, spare parts, etc., refer to Sections 4.2.4 and 4.2.5.

Table 5.33
Chlorine Dosage Requirements

Type of Treatment	Dosage (mg/L)
Raw Wastewater (Fresh)	6 – 15
Raw Wastewater (Septic)	12 – 25
Primary Effluent	5 – 20
Activated Sludge Plant Effluent	2 – 8
Trickling Filter Plant Effluent	3 – 10
RBC Plant Effluent	3 – 10
Tertiary Filtration Effluent	2 – 6
Nitrified Effluent	2 – 6

5.19.2.2.2. Odour Control
Should odour control be a critical factor, additional capacity of a pre-chlorination system to the extent of about 80% of the raw wastewater chlorine demand shall be required during the warm summer days. It is not desirable to split the functions of the chlorinators, especially for large plants. One group shall be designed for pre-chlorination and another for disinfection. In the case of large plants, each group shall be interchangeable to facilitate a standby feature.

Pre-chlorination must be accompanied ahead of the first open structure in the plant and thereby reduce the escape of hydrogen sulphide gas into the atmosphere.

5.19.2.3. Other
For additional information regarding chlorine supply, methods of dosage control, storage and handling, piping and connections, etc., refer to Section 4.2.

5.19.3. Dechlorination
Dechlorination of effluent shall be considered when the receiving water is:
1. Considered to be highly important for the fishing industry; or

2. Ecologically sensitive to chlorine toxicity and susceptible to the adverse effects of chlorine residuals; or

3. Of public health importance.

The decisions regarding use of dechlorination shall be made on a case-by-case basis.

Dechlorination is especially recommended for situations where low coliform densities, as well as chlorine residuals, are jointly required. The most common dechlorination chemicals are sulphur compounds, particularly sulphur dioxide gas or aqueous solutions of sulphite or bisulphate. Pellet dechlorination systems are also available for small facilities. The type of dechlorination system should be carefully selected considering criteria including the following: type of chemical storage required, amount of chemical needed, ease of operation, compatibility with existing equipment, and safety.

5.19.3.1. Dosage

The dosage of dechlorination chemicals should depend on the residual chlorine in the effluent, the final residual chlorine limit, and the particular form of the dechlorinating chemical used. The most common dechlorinating agent is sulphite. The forms of the compound that are commonly used and yield sulphite (SO_2) when dissolved in water are presented in Table 5.34.

Table 5.34
Dechlorination Chemicals and Required Amounts

Dechlorination Chemical	Theoretical mg/L Required to Neutralize 1 mg/L Cl_2
Sulphur Dioxide (gas)	0.9
Sodium meta bisulphate (solution)	1.34
Sodium bisulphate (solution)	1.46

Theoretical values may be used for initial approximations, to size feed equipment with the consideration that under good mixing conditions 10% excess dechlorinating chemical is required above theoretical values. Excess sulphur dioxide may consume oxygen at a maximum of 1.0 mg dissolved oxygen for every 4.0 mg SO_2.

The liquid solutions come in various strengths. The solutions may need to be further diluted to provide the proper dose of sulphite.

5.19.3.2. Containers

Depending on the chemical selected for dechlorination, the storage containers will vary from gas cylinders, liquid in a 190 L drums or dry compounds. Dilution tanks and mixing tanks will be necessary when using dry compounds and may be necessary when using liquid compounds to deliver the proper dosage. Solution containers should be covered to prevent evaporation and spills.

5.19.3.3. Feed Equipment, Mixing, and Contact Requirements

In general, the same type of feeding equipment used for chlorine gas may be used with minor modifications for sulphur dioxide gas. However, the manufacturer should be contacted for specific equipment recommendations. No equipment should be alternately used for the two gases. The common type of dechlorination feed equipment utilizing sulphur compounds include vacuum solution feed of sulphur dioxide gas and a positive displacement pump for aqueous solutions of sulphite or bisulphate.

The selection of the type of feed equipment utilizing sulphur compounds shall include consideration of the operator safety and overall public safety relative to the wastewater treatment plant's proximity to populated areas and the security of gas cylinder storage. The selection and design of sulphur dioxide feeding equipment shall take into account that the gas reliquifies quite easily. Special precautions must be taken when using ton containers to prevent reliquifaction.

Where necessary to meet the operating ranges, multiple units shall be provided for adequate peak capacity and to provide a sufficient low feed rate on turn down to avoid depletion of the dissolved oxygen concentrations in the receiving waters.

The dechlorination reaction with free or combined chlorine will generally occur within in 15 to 20 seconds. Mechanical mixers are required unless the mixing facility will provide the required hydraulic turbulence to assure thorough and complete mixing. The high solubility of SO_2 prevents it from escaping during turbulence.

A minimum of 30 seconds for mixing and contact time shall be provided at the design peak hourly flow or maximum pumping rate. A suitable sampling point shall be provided downstream of the contact zone. Consideration shall be given to a means of reaeration to assure maintenance of an acceptable dissolved oxygen concentration in the stream following sulphonation.

5.19.3.4. Housing Requirements

5.19.3.4.1. Feed and Storage Rooms

The requirements for housing SO_2 gas equipment should follow the same guidelines as used for chlorine gas.

When using solutions of the dechlorinating compounds, the solutions may be stored in a room that meets the safety and handling requirements set forth in Section 4.2.5. The mixing, storage, and solution delivery areas must be designed to contain or route solution spillage or leakage away from traffic areas to an appropriate treatment unit.

5.19.3.4.2. Protective and Respiratory Gear

The respiratory protection equipment is the same as used for chlorine (see Section 4.2.4). Leak repair kits of the type used for chlorine gas that are equipped with gasket material suitable for service with sulphur dioxide gas may be used. For additional safety considerations see Section 4.2.4.

5.19.3.5. Sampling and Control

Facilities shall be included for sampling the dechlorinated effluent for residual chlorine. Provisions shall be made to monitor for dissolved oxygen concentration after sulphonation when required by the DOEC.

Provisions shall be made for manual or automatic control of sulphonator feed rates based on chlorine residual measurement or flow.

5.19.3.6. Activated Carbon

Granular activated carbon may also be used to dechlorinate wastewater effluent. The dechlorination reaction is dependent on the chemical state of the free chlorine, chlorine concentration and flowrate, physical characteristics of the carbon, and the wastewater characteristics.

Dechlorination usually is accomplished in fixed downflow beds using gravity or pressure type filters. Regular backwashing is necessary to preserve dechlorination efficiency.

Suggested design criteria for a wastewater dechlorination activated carbon system, based on potable water application., include a wastewater application rate of 2 L/m^2s, an empty bed contact time of 15 to 20 minutes with an influent free residual of 3 to 4 mg/L, and an effective carbon bed life of at least 3 years.

5.19.4. UV Disinfection

The following sections describe factors that affect the performance of UV disinfection systems. Systems should be designed to account for these factors.

5.19.4.1. UV Transmission

UV light's ability to penetrate wastewater is measured with a spectrophotometer using the same wavelength (254 nm) that is produced by germicidal lamps. This measurement is called the percent Transmission or Absorbance and it is a function of all the factors that absorb or reflect UV light. As the percent transmission gets lower (higher absorbance) the ability of the UV light to penetrate the wastewater and reach target organisms decreases. The system designer must obtain samples of the wastewater during the worst conditions or carefully attempt to calculate the minimum expected UV transmission by testing wastewater from plants, which have a similar influent and treatment process. The designer must also strictly define the disinfection limits since they determine the magnitude of the UV dose required.

5.19.4.2. Wastewater Suspended Solids

Some of the suspended solids in wastewater will absorb or reflect the UV light before it can penetrate the solids to kill any occluded organisms. UV light can penetrate into suspended solids with longer contact times and higher intensities, but there is still a limit to the ability to kill the microorganisms. UV systems must be designed based on maximum effluent SS levels.

5.19.4.3. Design Flowrate and Hydraulics

The number of microorganisms that are inactivated within a UV reactor is a function of the multiplication of the average intensity and residence time, as per the following equation.

$$D = I \times t$$

As the flowrate increases the number or size of the UV lamps must be proportionally increased to maintain the same disinfection requirements. An UV disinfection system must be designed for worst-case conditions. The minimum dosage occurs at the maximum flowrate and end of lamp life.

5.19.4.4. Level Control

The height of the wastewater above the top row of UV lamps must be rigidly controlled by a flap gate or weir for all flowrates. The UV system must be designed for the maximum flowrate. This is especially important if the wastewater treatment plant receives storm water runoff. The UV system must also be designed to operate at the maximum flowrate. During low flow periods, the wastewater has a greater chance to warm up around the quartz sleeves and produce deposits on the sleeves. There is also the possibility of exposing the quartz sleeves to the air. Because the lamps are warm, any compounds left on the sleeves will bake onto them. Water splashing onto these exposed sleeves will also result in UV absorbing deposits. When the flow returns to normal, some of the water passing through the UV unit may not be properly disinfected. The designer must be very careful with the selection of the flow control device for the above situation. Both flow gates and weirs may be used for level control.

5.19.4.5. Iron Content

Iron can affect the UV disinfection by absorbing UV light. Dissolved iron, iron precipitate on quartz sleeves, and adsorption of iron by suspended solids, bacterial floc and other organic compounds, all decrease UV transmittance. Wastewater with iron levels greater than 0.3 mg/L may require pre-treatment to attain the desired disinfection level.

5.19.4.6. Wastewater Hardness

Calcium and magnesium salts, which are generally present in water as bicarbonates or sulphates, cause water hardness. Hard water will precipitate on any warm or hot surface. Since the optimum operating temperature of the low-pressure mercury lamp is 40°C, the surface of the protective quartz sleeve will be warm. It will create a molecular layer of warm water where calcium and magnesium salts can be precipitated. These precipitates will prevent some of the UV light from entering the wastewater.

Waters which approach or are above 300 mg/L of hardness may require pilot testing of a UV system. This is especially important if very low flows or no flow situations are expected, because they allow the water to warm up around the quartz sleeves and produce excessive coating.

5.19.4.7. Wastewater Sources

Periodic influxes of industrial wastewater may contain UV absorbing organic compounds, iron or hardness, any of which may affect UV performance. Industries discharging wastes that contain such materials may be required to pre-treat their wastewater.

Low concentrations of dye may be too diluted to be detected without using a spectrophotometer. Dye can readily absorb UV light thereby preventing UV disinfection.

5.19.4.8. UV Lamp Life

Low-pressure mercury lamps are rated for 9000 hours of continuous use. Rated average useful life is defined by the UV disinfection industry as the elapsed operating time under essentially continuous operation for the output to decline to 60% of the output the lamp had at 100 hours. The UV system must be designed so that the minimum required dose or intensity is available at the end of lamp life.

Power costs and lamp replacement costs are the two main factors affecting UV maintenance expenditures. Therefore, UV lamps should only be replaced of no other cause for not meeting the disinfection requirements can be found. Examples of other causes are quartz sleeve fouling, decreased levels of UV transmission, or increased levels of suspended solids in the wastewater.

5.19.4.9. UV System Configuration and Redundancy

Once the number of lamps required to meet the required disinfection permit levels has been determined, a system configuration must be developed. This configuration must meet operational requirements such as plant flow variations and redundancy requirements. Redundancy helps insure that the UV system can continue to operate and meet disinfection permits in spite of a subsystem or component failure. It allows regularly scheduled maintenance such as quartz cleaning to be performed at any time.

5.19.5. Ozone

Ozone Generation

Ozone may be produced from either an air or an oxygen gas source. Generation units shall be automatically controlled to adjust ozone production to meet disinfcetion requirements.

5.19.5.1. Dosage

The ozone demand in the wastewater must be satisfied, as evidenced by the presence of an ozone residual, before significant disinfection takes place. Below this dosage there is reduction of oxygen-consuming material.

Because of the form of ozone and its short life, it is necessary that it be step-fed into the wastewater to provide the contact period needed to accomplish disinfection.

Effectiveness of ozone as a disinfectant is relatively independent of pH and temperature values, although a pH of 6.0 to 7.0 appears to be the most favourable range. A dosage of 5 to 8 mg/L is needed to accomplish disinfection of secondary effluent. The amount and characteristics of suspended solids present in the secondary effluent can be used to determine ozone dosage empirically:

$$Ozone\,Dosage = 1.5 + 0.38\,TSS$$

5.19.5.2. Design Considerations

5.19.5.2.1. Feed Equipment

Ozone dissolution is accomplished through the use of conventional gas diffusion equipment, with appropriate consideration of materials. If ozone is being produced from air, gas preparation equipment (driers, filters, compressors) is required. If ozone is being produced from oxygen, this equipment may not be needed, as a clean dry pressurized gas supply will be available.

Where ozone capacities of 500 kg/day or less are required, air feed is preferred. Modification of the single-pass air feed system should be considered in determining the most economic system for application in wastewater treatment.

5.19.5.2.2. Air Cleaning

Removal of foreign matter such as dirt and dust is essential for optimum performance and life of an ozone device. For small units, cartridge-type impingement filters may be economical. For larger operations, electrical precipitator or combination filters are preferred.

5.19.5.2.3. Compression

Positive displacement rotary-type compressors are preferred for large installations. Internally lubricated units should not be used since oil vapour will permanently impair the water-adsorptive capacity of the driers. Need for standby capacity and flexibility of operation requires the installation of several blower units

The required compressor rating will depend on the pressure drop through the entire system. Generally, a 70 kN/m^2 pressure is necessary to force the air through the coolers, driers, ozonation devices, and the 4.5 to 6 m head of water in the mixing and contact system.

5.19.5.2.4. Cooling and Drying

Pretreatment for reducing moisture in the feed gas stream shall be required.

5.19.5.2.5. Injection, Mixing and Contact

Intimate mixing of an ozone-enriched air stream with the wastewater as well as maintaining contact for an adequate period of time is essential. The major problems to be considered are satisfying the ozone demand, the rapid rise of the gas to the liquid surface of the contact chamber and escape of ozonated air bubbles, and the relatively short half-life of ozone. Consequently, where ozone contact beyond a few minutes is needed, the ozonated feed stream is staged with the amount of ozone for each stage set at a level that can be consumed usefully.

5.19.5.2.6. Controls

The design engineer should be cognizant of the fact that ozone is a toxic gas, and that if compressed oxygen is used as the feed gas, special provisions must be met in its handling and storage. The ozonation process involves a series of mechanical and electrical units that require appropriate maintenance and repair and are susceptible to the same malfunctions as are all such pieces of equipment. Standby capacity normally is provided in all essential components. Information can be obtained from the equipment manufacturer on the metering and alarm systems needed for continuous process monitoring and warning of failure in any element of the process.

5.19.5.2.7. Piping and Connections

For ozonation systems, the selection of material should be made with due consideration for ozone's corrosive nature. Copper or aluminum alloys should be avoided. Only material at least as corrosion-resistant to ozone as Grade 304L stainless steel should be specified for piping containing ozone in non-submerged applications. Unplasticized PVC, Type 1, may be used in submerged piping, provided the gas temperature is below 60°C and the gas pressure is low.