Environment Canada   Environnement Canada

Atlantic Canada Wastewater Guidelines Manual

for Collection, Treatment, and Disposal

2006

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Chapter 1 APPROVAL REQUIREMENTS AND PROCEDURES

1.1 APPLICATION FOR APPROVAL

The regulatory authorities require that application for approval be made in writing by a person responsible for the construction, modification, or operation of sewage works. The application shall be submitted to the appropriate regulatory agency.

An application for the construction or modification of sewage works shall include engineering reports, plans and specifications and all other information which the regulatory agencies may require.

Approval in principle of preliminary reports and plans (concept approval) shall not constitute official approval. No approval for construction or modification can be issued until final detailed plans and specifications have been submitted to the regulatory agency and found to be satisfactory. Such works shall not be undertaken until an official "Certificate of Approval" or “Approval/Permit to Construct” bearing the necessary signatures has been issued by the Minister of the appropriate regulatory agency.

All final reports, plans and specifications should be submitted at least 90 days prior to the start of the construction or modification. The reports, plans or specifications shall be stamped with the seal and signature of the designing engineer, licensed to practice in the Province of application.

The application shall include sufficient design information and one complete set of plans and specifications submitted directly to the appropriate regulatory agency.

Engineering services are performed in four (4) steps:

a) preliminary evaluation;

b) pre-design report;

c) preparation of construction plans, specifications, contractual documents and design report; and

d) construction compliance, inspection, administration and acceptance, and submission of a post-construction report.

These services are generally performed by engineering firms in private practice but may be executed by municipal or provincial agencies.

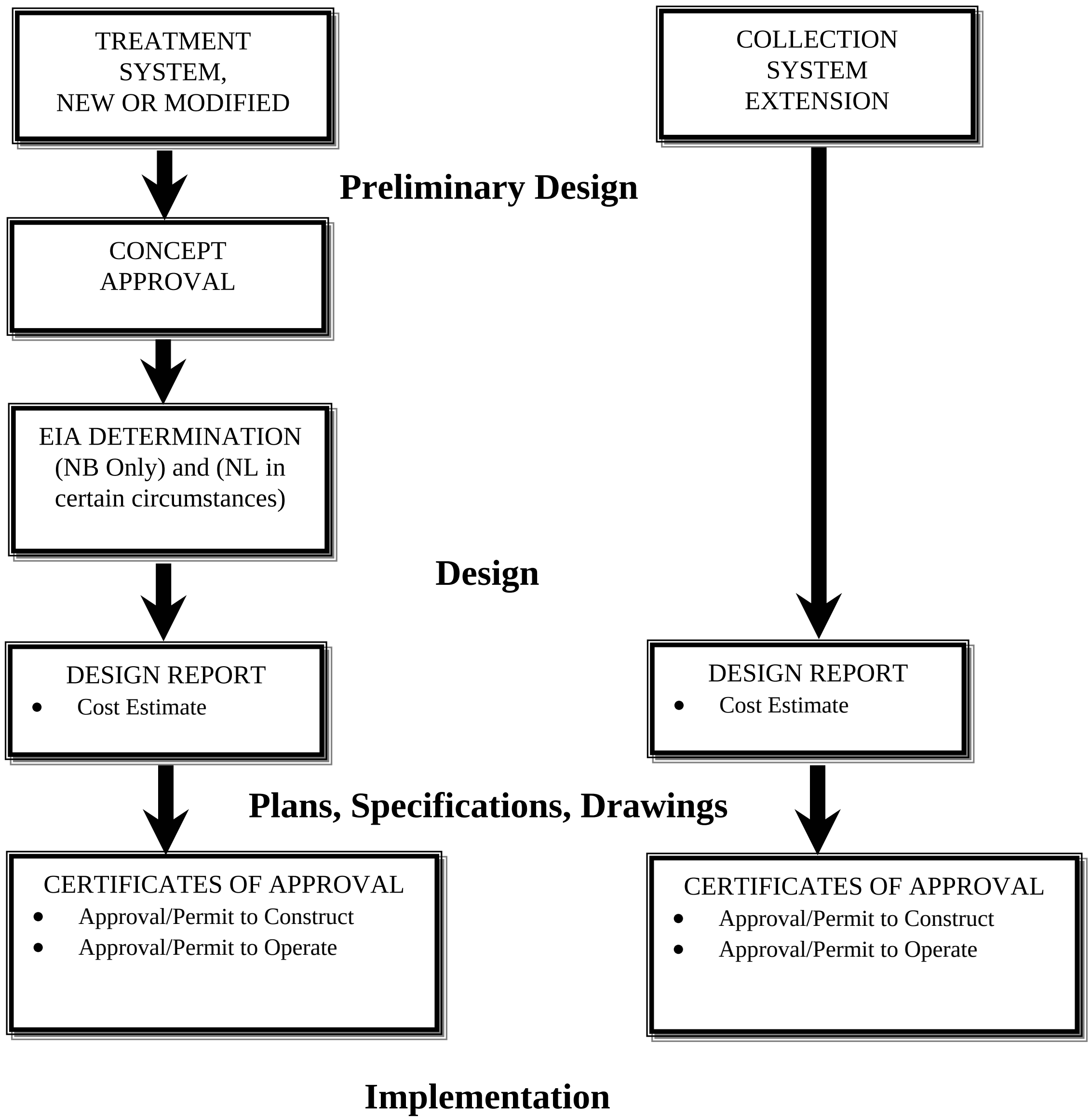
The overall approvals process is outlined in Figure 1.1.

1.2 PRE-DESIGN EVALUATION

A pre-design evaluation shall broadly:

a) describe existing problems;

FIGURE 1.1
APPROVALS PROCESS



Note: Nova Scotia and Prince Edward Island issue two separate approvals for construction and operation. Newfoundland and Labrador issues a permit to construct and permit to operate. New Brunswick issues a certificate of approval to construct and a certificate of approval to operate after 6 months of continuous and successful operation.

b) assess a receiving waters' assimilative capacity;

c) describe design parameters;

d) consider methods for alternate solutions including site and/or route selection;

e) estimate capital and annual operating costs; and

f) outline steps for further project implementation including applications for grants-in-aid and approval by regulatory agencies.

1.2.1 Effluent Discharge Requirements

In the case of wastewater treatment plant effluent, discharge requirements will be set by regulatory agencies. These requirements may be as a result of receiving water studies or they may be governed by a pre-determined discharge policy. Regulatory agencies having jurisdiction should be contacted prior to the start of the pre-design study to determine whether discharge parameters will be set by the regulatory authority or if a receiving water study will be utilized for setting the discharge parameters.

The Canadian Council of Ministers of the Environment (CCME) comprises environment ministers from the federal, provincial and territorial governments. The CCME is currently developing a Canada-wide Strategy for the management of municipal wastewater effluents. Completion of the Strategy is expected in 2007. The objective of the Strategy is to ensure that the release of wastewater effluent does not pose unacceptable risks to citizens and the environment. The Strategy will include the development of a harmonized regulatory framework and national standards for specific pollutants. Further information can be found on the CCME website at: www.ccme.ca.

1.2.2 Flow Gauging and Wastewater Characterization Studies

Prior to the preparation of a pre-design report on an existing sewerage system, a comprehensive flow gauging and wastewater characterization study should be conducted. This will aid in gaining a better understanding of important design criteria such as flow rates and variations and wastewater composition.

1.2.3 Infiltration/Inflow Investigations

Prior to the preparation of a pre-design report on an existing sewerage system, a comprehensive infiltration/inflow investigation should be conducted. This will give the designers a better indication of extraneous flow contributions, as well as aid in design solutions, (i.e. the potential for reducing flows at an existing plant).

1.2.4 At-Source Control

Urban wastewater may be composed of many metals/chemicals that are discharged to the sewer system, but may not be treated by conventional treatment, or the level of treatment proposed. This can result in concentrating these constituents into the treatment plant sludge, receiving water, and sediments near the plant outfall.

To protect worker health, collection and treatment infrastructure, and the environment, design of collection and treatment systems must address by-laws and enforcement which keep such materials out of the system (at-source control). Decreased levels of contaminants in wastewater sludge may also result in a saleable commodity that can have economic benefits.

1.3 PRE-DESIGN REPORT

Pre-design reports are necessary in order to obtain a concept approval from the appropriate regulatory agency. The pre-design report assembles basic information; presents design criteria and assumptions; examines alternate projects with preliminary layouts and cost estimates; describes financing methods giving anticipated charges for users; reviews organizational and staffing requirements; offers a conclusion with a proposed project for client consideration; and outlines official actions and procedures to implement the project.

The concept, factual data and controlling assumptions and considerations for the functional planning of sewage facilities are presented for each process unit and for the whole system. These data form the continuing technical basis for detail design and preparation of construction plans and specifications.

Architectural, structural, mechanical and electrical designs are usually excluded. Sketches may be desirable to aid in presentation of a project. Outline specifications of process units, special equipment, etc., are occasionally included.

1.3.1 Purpose

A pre-design report for a proposed project is used:

a) by the municipality for a description, cost estimates, financing requirements, user commitments, findings, conclusions and recommendations, as a guide to adopt a well-defined project;

b) by the regulatory agency for examination of process operation, control, safety and performance directed to maintenance of water quality when facilities are discharging processed sewage;

c) by investment groups and government funding agencies to evaluate the "quality" of the proposed project with reference to authorization and financing; and

d) by news media for telling a story.

1.3.2 Relation to a Comprehensive Study

The pre-design report for a specific project should be an "outgrowth" of and consistent with an area wide and drainage basin comprehensive study or master plan.

1.3.3 Contents

The pre-design report, to be acceptable for review and approval, must:

a) develop predicted population;

b) establish a specific service area for immediate consideration and indicate possible extensions;

c) present reliable measurements of flow and analyses of wastewater constituents as a basis of process design;

d) estimate costs of immediately proposed facilities;

e) present a reasonable method of financing and show typical financial commitments;

f) suggest an organization and administrative procedure;

g) consider operational requirements with regard to protection of receiving water quality;

h) reflect local bylaws and Federal/Provincial regulations;

i) present summarized findings, conclusions and recommendations for the owner's guidance;

j) include a site plan. This plan must indicate locations of residences, private and public water supplies, recreational areas, watercourses, zoning, floodplains and other areas of concern when siting sewage collection and treatment facilities;

k) identify existing problems; including combined sewer overflows (CSO's) and sanitary sewer overflows (SSO's) and proposed remedial measures to correct any of the problems.

l) identify existing and potential receiving water uses; and

m) identify possible treatment plant locations.

1.3.4 Concept and Guidance for Plans and Specifications

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

1.3.5 Format for Content and Presentation

It is urged that the following subsections be utilized as a guideline for content and presentation of the project pre-design report to the Provincial Regulatory Agency for review and approval.

1.3.5.1 Title

The Wastewater Facilities Pre-Design Report - collection, conveyance, processing and discharge of wastewater.

1.3.5.2 Letter of Transmittal

A one page letter typed on the firm's letterhead and bound into the report should include:

a) submission of the report to the client;

b) statement of feasibility of the recommended project;

c) acknowledgement to those giving assistance; and

d) reference to the project as outgrowth of approved or "master" plan.

1.3.5.3 Title Page

a) title of project;

b) municipality, county, etc.;

c) names of officials, managers, superintendents;

d) name and address of firm preparing the report; and

e) seal and signature of professional engineer(s) in charge of the project.

1.3.5.4 Table of Contents

a) Section headings, chapter headings and sub-headings;

b) maps;

c) graphs;

d) illustrations, exhibits;

e) diagrams; and

f) appendices.

1.3.5.5 Summary

Highlight, very briefly, what was found from the study.

1.3.5.5.1 Findings

a) population-present, design (when), ultimate;

b) land use and zoning - portion per residential, commercial, industrial, greenbelt, etc;

c) wastewater characteristics and concentrations - portions of total hydraulic, organic and solid loading attributed to residential commercial and industrial fractions;

d) collection system projects - immediate needs to implement recommended project, deferred needs to complete recommended project and pump stations, force mains, appurtenances, etc.

e) selected process - characteristics of process and characteristics of output.

f) receiving waters - existing water quality and quantity, downstream water uses and impact of project on receiving water;

g) proposed project - total project cost, total annual expense requirement for: debt service; operation, personnel and operation, non-personnel;

h) environmental assessment of selected process;

i) energy requirements - quantities, costs and forms;

j) finances - indicate financing requirements and typical annual charges;

k) organization - administrative control necessary to implement project, carry through to completion and operate and maintain wastewater facility and system; and

l) changes - alert client to situations that could alter recommended project.

1.3.5.5.2 Conclusions

Project, or projects, recommended to client for immediate construction, suggested financing program, etc.

1.3.5.5.3 Recommendations

Summarized, step-by-step actions, for the client to follow in order to implement conclusions:

a) acceptance of report;

b) adoption of recommended project;

c) submission of report to regulatory agencies for review and approval;

d) authorization of engineering services for approved project (construction plans, specifications, contract documents, etc.);

e) legal services

f) enabling ordinances, resolutions, etc., required;

g) adoption of sewer-use ordinance;

h) adoption of operating rules and regulations;

i) financing program requirements;

j) organization and administration (structure, personnel, employment, etc.);

k) time schedules - implementation, construction, completion dates, reflecting applicable hearings, stipulations, abatement orders.

1.3.5.6 Introduction

1.3.5.6.1 Purpose

Reasons for report and circumstances leading up to report.

1.3.5.6.2 Scope

Coordination of recommended project with approved comprehensive master plan and guideline for developing the report.

1.3.5.7 Background

Present only appropriate past history.

1.3.5.7.1 General

a) existing area, expansion, annexation, inter-municipal service, ultimate area;

b) drainage basin, portion covered;

c) population growth, trends, increase during design life of facility (graph);

d) residential, commercial and industrial land use, zoning, population densities, industrial types and concentrations;

e) topography, general geology and effect on project;

f) meteorology, precipitation, runoff, flooding, etc. and effect on project; and

g) total period of time for which project is to be studied.

1.3.5.7.2 Economic

a) assessed valuation, tax structure, tax rates, portions for residential, commercial, industrial property.

b) employment from within and outside service area;

c) transportation systems, effect on commuter influx;

d) exempt property; churches and agricultural exhibition, properties and effect on project; and

e) costs of present water and wastewater services.

1.3.5.7.3 Regulations

a) existing ordinances, rules and regulations including defects and deficiencies, etc;

b) recommended amendments, revisions or cancellation and replacement;

c) sewer-use ordinance (toxic, aggressive, volatile, etc., substances);

d) surcharge based on volumes and concentration for industrial wastewaters;

e) existing contracts and agreements (inter-municipal, etc.); and

f) enforcement provisions including inspection, sampling detection, penalties, etc.

1.3.5.8 Hydraulic Capacity

The following flows for the design year shall be identified and used as a basis for design for sewers, lift stations, wastewater treatment plants, treatment units, and other wastewater handling facilities. Where any of the terms defined in this

Section are used in these design standards, the definition contained in this Section applies.

a. Design Average Flow

The design average flow is the average of the daily volumes to be received for the continuous 12 month period expressed as a volume per unit time. However, the design average flow for facilities having critical seasonal high hydraulic loading periods (e.g., recreational areas, campuses, industrial facilities) shall be based on the daily average flow during the seasonal period.

b. Design Maximum Day Flow

The design maximum day flow is the largest volume of flow to be received during a continuous 24 hour period expressed as a volume per unit time.

c. Design Peak Hourly Flow

The design peak hourly flow is the largest volume of flow to be received during a one hour period expressed as a volume per unit time.

d. Design Peak Instantaneous Flow

The design peak instantaneous flow is the instantaneous maximum flow rate to be received.

e. Design Minimum Day Flow

The design minimum day flow is the smallest volume of flow to be received during a 24 hour period during dry weather when infiltration/inflow are at a minimum, expressed as a volume per unit time.

1.3.5.9 Investigative Considerations - Existing Facilities Evaluation

1.3.5.9.1 Existing Collection System

a) Inventory of existing sewers;

b) isolation from water supply wells;

c) adequacy to meet project needs (structural condition, hydraulic capacity tabulation);

d) gauging and infiltration tests (tabulate);

e) overflows and required maintenance, repairs and improvements;

f) outline repair, replacement and storm water separation requirements;

g) evaluation of costs for treating infiltration/inflow versus costs for rehabilitation of system;

h) establish renovation priorities, if selected;

i) present recommended annual program to renovate sewers; and

j) indicate required annual expenditure.

1.3.5.9.2 Existing Treatment Plant

a) area for expansion;

b) surface condition;

c) subsurface conditions;

d) isolation from habitation;

e) isolation from water supply structures;

f) enclosure of units, winter conditions, odour control, landscaping, etc.; and

g) flooding (predict elevation of 25 and 100 year flood stage). Including climate change and an increase in sea level.

1.3.5.9.3 Existing Process Facilities

a) capacities and adequacy of units (tabulate);

b) relationship and/or applicability to proposed project;

c) age and condition;

d) adaptability to different usages;

e) structures to be retained, modified or demolished; and

f) outfall.

1.3.5.9.4 Existing Wastewater Characteristics

a) water consumption (from records) total, unit, industrial;

b) wastewater flow pattern, peaks, total design flow;

c) physical, chemical and biological characteristics and concentrations; and

d) residential, commercial, industrial, infiltration fractions, considering organic solids, toxic aggressive, etc., substances; tabulate each fraction separately and summarize.

1.3.5.10 Proposed Project

1.3.5.10.1 Collection System

a) inventory of proposed additions;

b) isolation from water supply well, reservoirs, facilities, etc;

c) area of services;

d) unusual construction problems;

e) utility interruption and traffic interference;

f) restoration of pavements, lawns, etc.; and

g) basement flooding prevention during power outage.

1.3.5.10.2 Site Requirements

Comparative advantages and disadvantages as to cost, hydraulic requirements, flood control, accessibility, enclosure of units, odour control, landscaping, etc., and isolation with respect to potential nuisances and protection of water supply facilities.

1.3.5.10.3 Wastewater Characteristics

a) character of wastewater necessary to insure amenability to process selected;

b) need to pretreat industrial wastewater before discharge to sewers;

c) portion of residential, commercial, industrial wastewater fractions to comprise projected growth.

1.3.5.10.4 Receiving Water Considerations and Assimilative Capacity

a) wastewater discharges upstream;

b) receiving water base flow (utilize critical flow as specified by approving agency);

c) characteristics (concentrations) of receiving waters;

d) downstream water uses including water supply, recreation, agricultural, industrial, etc;

e) impact of proposed discharge on receiving waters;

f) tabulate assimilative capacity requirements;

g) listing of effluent characteristics; and

h) tabulation and correlation of plant performance versus receiving water requirements.

1.3.5.11 Alternatives

Alternatives should consider such items as regional solution, optimum operation of existing facilities, flow and waste reduction, location of facilities, phased construction, necessary flexibility and reliability, sludge disposal, alternative treatment sites, alternative processes and institutional arrangements.

1.3.5.11.1 Alternate Process and Site

a) describe and delineate (line diagrams);

b) preliminary design for cost estimates;

c) estimates of project cost (total) dated, keyed to construction cost index, escalated, etc;

d) advantages and disadvantages of each;

e) individual differences, requirements, limitations;

f) characteristics of process output;

g) comparison of process performances;

h) operation and maintenance expenses;

i) annual expense requirements (tabulation of annual operation, maintenance, personnel, debt obligation for each alternate), and

j) environmental assessment of each.

1.3.5.12 Selected Process and Site

a) identify and justify process and site selected;

b) adaptability to future needs;

c) environmental assessment;

d) outfall location; and

e) describe immediate and deferred construction.

1.3.5.13 Project Financing

a) review applicable financing methods;

b) effect of Provincial and Federal funding;

c) assessment by front metre, area unit or other benefit;

d) charges by connection, occupancy, readiness-to-serve, water consumption, industrial wastewater discharge, etc;

e) existing debt service requirements;

f) annual financing and bond retirement schedule;

g) tabulate annual operating expenses;

h) show anticipated typical annual charge to user and non-user; and

i) show how representative properties and users are to be affected.

1.3.5.14 Legal and Other Considerations

a) needed enabling legislation, ordinances, rules and regulations;

b) contractual considerations for inter-municipal cooperation;

c) public information and education; and

d) statutory requirements and limitations.

1.3.5.15 Appendices: Technical Information and Design Criteria

1.3.5.15.1 Collection System

a) design tabulations - flow, size, velocities, etc;

b) regulator or overflow design;

c) pump station calculations, including energy requirements;

d) special appurtenances;

e) stream crossings; and

f) system map (report size).

1.3.5.15.2 Process Facilities

a) criteria selection and basis;

b) hydraulic and organic loadings - minimum, average, maximum and effect;

c) unit dimensions;

d) rates and velocities;

e) detentions;

f) concentrations;

g) recycle;

h) chemical additive control;

i) physical control;

j) removals, effluent concentrations, etc. Include a separate tabulation for each unit to handle solid and liquid fractions;

k) energy requirement; and

l) flexibility.

1.3.5.15.3 Process Diagrams

a) process configuration, interconnecting piping, processing, flexibility, etc;

b) hydraulic profile;

c) organic loading profile;

d) solids control system;

d) solids profile; and

e) flow diagram with capacities, etc.

1.3.5.15.4 Space for Personnel, Laboratories and Records

1.3.5.15.5 Chemical Control

a) processes needing chemical addition;

b) chemicals and feed equipment; and

c) tabulation of amounts and unit and total costs.

1.3.5.15.6 Support Data

a) outline unusual specifications, construction materials and construction methods;

b) maps, photographs, diagrams (report size);

c) other.

1.3.6 Supplemental Information

1.3.6.1 Treated Effluent To Land

In addition to the required pre-design report, the designer shall include supplemental information, as outlined below. This information shall include any material that is pertinent about the location, geology, topography, hydrology, soils, areas for future expansion, and adjacent land use.

1.3.6.1.1 Location

The following supplement information is required to be submitted with the pre-design report.

1. A copy of the topographic map of the area showing the exact boundaries of the proposed application area.

2. A topographic map of the total area owned by the applicant at a scale of approximately 1: 10 000. It should show all buildings, the waste disposal system, the spray field boundaries and the buffer zone. An additional map should show the spray field topography in detail with a contour interval of 0.5 m and include buildings and land use on adjacent lands within 400 m of the project boundary.

3. All water supply wells which might be affected shall be located and identified as to use; e.g., potable, industrial, agricultural, and class of ownership; e.g., public, private, etc.

4. All abandoned wells, shafts, etc., shall be located and identified. Pertinent information therein shall be furnished.

5. Separation distances shall comply with requirements of section 4.3.2

1.3.6.1.2 Geology

1. The geologic formations (name) and the rock types at the site.

2. The degree of weathering of the bedrock.

3. The local bedrock structure including the presence of faults, fractures and joints.

4. The character and thickness of the surficial deposits (residual soils and glacial deposit).

5. In limestone terrain, additional information about solution openings and sinkholes is required.

6. The source of the above information must be indicated.

1.3.6.1.3 Hydrology

1. The depth to seasonal high water table (perched and/or regional) must be given, including an indication of seasonal variations. Static water levels must be determined at each depth for each aquifer in the depth under

concern. Critical slope evaluation must be given to any differences in such levels.

2. The direction of groundwater movement and the point(s) of discharge must be shown on one of the attached maps.

3. Chemical analyses indicating the quality of groundwater at the site must be included.

4. The source of the above data must be indicated.

5. The following information shall be provided from existing wells and from such test wells as may be necessary:

   a. Construction details - where available; Depth, well log, pump capacity, static levels, pumping water levels, casing, grout material, and such other information as may be pertinent.

   b. Groundwater quality: e.g., Nitrates, total nitrogen, chlorides, sulphates, pH, alkalinities, total hardness, coliform bacteria, etc.

6. A minimum of one groundwater monitoring well must be drilled for the protection of potable water wells or as determined by the Regulatory agency have jurisdiction, in each dominant direction of groundwater movement and between the project site and public well(s) and/or high-capacity private wells, with provision for sampling at the surface of the water table and at 1.5 m below the water table at each monitoring site. The location and construction of the monitoring well(s) must be approved by the regulatory authority. These may include one or more of the test wells where appropriate.

1.3.6.1.4 Soils

1. A soils map of the spray field should be furnished, indicating the various soil types. This may be included on the large-scale topographic map. Soils information can normally be secured through the Federal Department of Energy Mines and Resources, the Federal Department of Agriculture, or the applicable provincial department.

2. The soils should be named and their texture described.

3. Slopes and agricultural practice on the spray field are closely related. Slopes on cultivated fields should be limited to 4%.

   Slopes on sodded fields should be limited to 8%. Forested slopes should be limited to 8% for year-round operation, but some seasonal operation slopes up to 14% may be acceptable.

4. The thickness of soils should be indicated. Method of determination should be included.

5. Data should be furnished on the exchange capacity of the soils. In case of industrial wastes particularly, this information must be related to special characteristics of the wastes.

6. Information must be furnished on the internal and surface-drainage characteristics of the soil materials. This includes the soil's infiltration capacity and permeability.

7. Proposed application rates should take into consideration the drainage and permeability of the soils, the discharge capacity, and the distance to the water table.

1.3.6.1.5 Agricultural Practice

1. The present and intended soil-crop management practices, including forestation, shall be stated.

2. Pertinent information shall be furnished on existing drainage systems.

3. When cultivated crops are anticipated, the kinds used and the harvesting frequency should be given; the ultimate use of the crop should also be given. See Section 10.3.3.4 for crop considerations.

1.3.6.1.6 Adjacent Land Use

1. Present and anticipated use of the adjoining lands, up to 400m from the site, must be indicated. This information can be provided on one of the maps and may be supplemented with notes.

2. The plan shall show existing and proposed screens, barriers, or buffer zones to prevent blowing spray from entering adjacent land areas.

3. If expansion of the facility is anticipated, the lands which are likely to be used for expanded spray fields must be shown on the map.

1.4 DETAILED DESIGN DOCUMENTATION

1.4.1 General

Upon obtaining a concept approval the owner or his/her representative must prepare and submit detailed design documentation. This includes a Design Report, plans, specifications and contractual documents, and any applications for approval required by the regulatory agency with jurisdiction over the proposed project.

1.4.2 Design Report

The Design Report shall contain detailed design calculations for each unit or process of the wastewater treatment or collection facility. The design report shall also address operational and maintenance issues for that particular facility.

1.4.2.1 Format for Content and Presentation

It is urged that the following subsection be utilized as a guideline for content and presentation of the project Design Report to the appropriate regulatory agency for review and approval.

1.4.2.1.1 Title

The Wastewater Facilities Design Report - collection, conveyance, processing and discharge of wastewater.

1.4.2.1.2 Letter of Transmittal

A one page letter typed on the firm's letterhead and bound into the report should include:

a) submission of the report to the client;

b) acknowledgement to those giving assistance; and

c) reference to the project as outgrowth of approved or "master" plan.

1.4.2.1.3 Title Page

a) title of project;

b) municipality, county, etc;

c) names of officials, managers, superintendents;

d) name and address of firm preparing the report; and

e) seal and signature of professional engineer(s) in charge of the project.

1.4.2.1.4 Table of Contents

a) section headings, chapter headings and sub-headings

b) maps;

c) graphs;

d) illustrations, exhibits;

e) diagrams; and

f) appendices.

1.4.2.1.5 Collection System

a) detailed design tabulations - flow, size, velocities, etc;

b) regulator or overflow design calculations;

c) detailed pump station calculations, including energy requirements;

d) special appurtenances;

e) stream crossings; and

f) system map (report size).

1.4.2.1.6 Process Facilities

a) hydraulic and organic loadings - minimum, average, maximum and effect;

b) detailed calculations used to determine:

- unit dimensions;
- rates and velocities;
- detentions;
- concentrations;
- recycle;
- removals, effluent concentrations, etc. Include a separate tabulation for each unit to handle solid and liquid fractions;
- energy requirement;
- flexibility; and

c) chemical requirements and control.

1.4.2.1.7 Process Diagrams

a) process configuration, interconnecting piping, processing, flexibility, etc;

b) hydraulic profile;

c) organic loading profile;

d) solids control system;

e) solids profile; and

f) flow diagram with capacities, etc.

1.4.2.1.8 Laboratory

a) physical and chemical tests and frequency to control process;

b) time for testing;

c) space and equipment requirements; and

d) personnel requirements - number, type, qualifications, salaries, benefits (tabulate).

1.4.2.1.9 Operation and Maintenance

a) routine and special maintenance duties;

b) time requirements;

c) tools, equipment, vehicles, safety, etc;

d) personnel requirements - number, type, qualifications, salaries, benefits, (tabulate); and

e) maintenance work space and storage.

1.4.2.1.10 Office Space for Administrative Personnel and Records

1.4.2.1.11 Personnel Service - Locker Room and Lunch Room

1.4.2.1.12 Chemical Control

a) process needing chemical addition;

b) chemicals and feed equipment; and

c) tabulation of amounts and unit and total costs.

1.4.2.1.13 Collection System Control

a) cleaning and maintenance;

b) regulator and overflow inspection and repair;

c) flow gauging;

d) industrial sampling and surveillance;

e) regulation enforcement;

f) equipment requirements;

g) trouble-call investigation; and

h) personnel requirements - number, type, qualifications, salaries, benefits (tabulate).

1.4.2.1.14 Control Summary

a) personnel;

b) equipment;

c) chemicals;

d) utilities - list power requirements of major units; and

e) summation.

1.4.2.1.15 Support Data

a) outline unusual specifications, construction materials and construction methods;

b) maps, photographs, diagrams (report size); and

c) other.

1.4.2.1.16 Appendices

Related data not necessary to an immediate understanding of the design report should be placed in the appendices.

1.4.3 Plans

1.4.3.1 General

All plans for sewage work shall bear a suitable title showing the name of the municipality, sewer district, or institution; and shall show the scale in appropriate units, the north point, date and the name of the engineer, his signature on an imprint of his registration seal.

The plans shall be clear and legible. They shall be drawn to scale which will permit all necessary information to be plainly shown. The size of the plans should be 570 x 817 mm (size A1 (21 x 33 in (size D)). Datum used should be indicated. Locations and logs of test borings, when made, shall be shown on the plans.

Detail plans shall consist of plan views, elevations, sections and supplementary views which, together with the specifications and general layouts, provide the working information for the contract and construction of the works. Include dimensions and geodetic elevations of structures, the location and outline form of equipment, location and size of piping, water levels and ground elevations.

1.4.3.2 Plans of Sewers

1.4.3.2.1 General Plans

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

a) Geographical Features

i) topography and elevations - existing or proposed streets and all streams or water surfaces shall be clearly shown. Contour lines at suitable intervals should be included;

ii) streams -the direction of flow in all streams and high and low water elevations of all water surfaces at sewer outlets and overflows shall be shown.

iii) boundaries - the boundary lines of the municipality, the sewer district or area to be sewered shall be shown.

b) Sewers

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

c) Identify sensitive areas and potential environment issues.

1.4.3.2.2 Detail Plans

Detail plans shall be submitted. Profiles should have a horizontal scale of not more than 1:500 and a vertical scale of not more than 1:50. Plans and profiles shall show:

a) location of streets and sewers;

b) line of ground surface, size, material and type of pipe, length between manholes, invert and surface elevation at each manhole and grade of sewer between each two adjacent manholes. All manholes shall be numbered on the plan and correspondingly numbered on the profile.

Where there is any question of the sewer being sufficiently deep to serve any residence, the elevation and location of the basement floor shall be plotted on the profile of the sewer which is to serve the house in question. The engineer shall state that all sewers are sufficiently deep to serve adjacent basements except where otherwise noted on the plans.

c) locations of all special features such as inverted siphons, concrete encasement, elevated sewers, etc;

d) all known existing structures both above and below ground which might interfere with the proposed construction, particularly water mains, gas mains, storm drains, etc.;

e) special detail drawings, made to a scale to clearly show the nature of the design, shall be furnished to show the following particulars:

(i) all stream crossings and sewer outlets, with elevations of the stream bed and of normal and extreme high and low water levels;

(ii) details of all special sewer joints and cross-sections; and

(iii) details of all sewer appurtenances such as manholes, lamp holes, inspection chambers, inverted siphons, regulators, tide gates and elevated sewers.

f) Details and plans of CSOs and treatment components according to the Regulatory Agency having jurisdiction.

1.4.3.3 Plans of Sewage Pumping Stations

1.4.3.3.1 Location Plan

A plan shall be submitted for projects involving construction or revision of pumping stations. This plan shall show the following:

a) the location and extent of the tributary area;

b) any municipal boundaries with the tributary area; and

c) the location of the pumping station and force main and pertinent elevations.

d) identify sensitive areas and potential environment issues.

1.4.3.3.2 Detail Plans

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

a) topography of the site;

b) existing pumping station;

c) proposed pumping station, including provisions for installation of future pumps;

d) elevation of high water at the site and maximum elevation of sewage in the collection system upon occasion of power failure;

e) maximum hydraulic gradient in downstream gravity sewers when all installed pumps are in operation; and

f) test borings and groundwater elevations.

g) Details and plans of CSOs and treatment components according to the Regulatory Agency having jurisdiction.

1.4.3.4 Plans of Sewage Treatment Plant

1.4.3.4.1 Location Plans

A plan shall be submitted, showing the sewage treatment plant in relation to the remainder of the system.

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

Identify sensitive areas and potential environment issues.

1.4.3.4.2 General Layout

Layouts of the proposed sewage treatment plant shall be submitted, showing:

a) topography of the site;

b) size and location of plant structures;

c) schematic flow diagram showing the flow through various plant units;

d) piping, including any arrangements for by-passing individual units. Materials handled and direction of flow through pipes shall be shown;

e) hydraulic profiles showing the flow of sewage, supernatant, mixed liquor and sludge; and

h) test borings and ground water elevations.

1.4.3.4.3 Detail Plans

a) Location, dimensions and elevations of all existing and proposed plant facilities;

b) elevations of high and low water level of the body of water to which the plant effluent is to be discharged;

c) type, size, pertinent features and manufacturer's rated capacity of all pumps, blowers, motors and other mechanical devices;

d) minimum, average and maximum hydraulic flow in profile; and

e) adequate description of any features not otherwise covered by specifications or engineer's report.

1.4.4 Specifications

Complete technical specifications for the construction of sewers, sewage pumping stations, sewage treatment plants and all appurtenances, shall accompany the plans.

The specifications accompanying construction drawings shall include, but not be limited to, all construction information not shown on the drawings which is necessary to inform the builder in detail of the design requirements as to the quality of materials and workmanship and fabrication of the project and the type, size, strength, operating characteristics and rating of equipment; allowable infiltration; the complete requirements for all mechanical and electrical equipment, including machinery, valves, piping and jointing of pipe; electrical apparatus, wiring and meters; laboratory fixtures and equipment; operating tools; construction materials; special filter materials such as stone, sand, gravel or slag; miscellaneous appurtenances, chemicals when used; instructions for testing materials and equipment as necessary to meet design standards; and operating tests for the completed works and component units. It is suggested that these performance tests be conducted at design load conditions wherever practical.

1.5 REVISIONS TO APPROVED PLANS

Any deviations from approved plans or specifications affecting capacity, flow or operation of units shall be approved in writing before such changes are made. Plans or specifications so revised should, therefore, be submitted well in advance of any construction work which will be affected by such changes, to permit sufficient time for review and approval. Structural revisions or other minor changes not affecting capacities, flows, or operation will be permitted during construction without approval. "As-built" plans clearly showing such alterations shall be submitted to the reviewing agency at the completion of the work.

1.6 CERTIFICATES OF APPROVAL

The Approval/Permit to Construct shall be issued prior to construction by the appropriate regulatory agency to the owner/operator only upon final approval of the Design report, plans, specifications and contract documents. The permit shall provide the owner/operator with the authority to proceed with the construction of that particular project. The Approval/Permit to Operate shall be issued to the owner/operator, prior to operation, by the appropriate regulatory agency only upon successful completion of construction, application for treatment plant classification and the naming of the treatment plant operator(s). The permit shall provide the owner/operator with the authority to proceed with the operation of that particular project. In the case of New Brunswick the Certificate of Approval to Operate is issued after a period of 6 months of continuous and successful operation.

1.7 OPERATION DURING CONSTRUCTION

Specifications shall contain a program for keeping existing treatment plant units in operation during construction of plant additions. Should it be necessary to take plant units out of operation, a shut-down procedure which will mitigate pollution effects on the receiving water or land, shall be reviewed and approved in advance by the appropriate reviewing agency(s).

1.8 OPERATING REQUIREMENTS

1.8.1 General

Any newly constructed sewerage system or treatment plant shall be put into operation only if it meets appropriate of the following criteria:

a) in the case of Nova Scotia and Prince Edward Island an “”Approval to Construct” and an “Approval to Operate” have been issued by the regulatory agency.

b) in the case of " Newfoundland and Labrador a “Permit to Construct” and a “Permit to Operate” has been issued by the regulatory agency to the owner/operator of the system or treatment plant.

c) in the case of New Brunswick, application for a Certificate of Approval to Operate has been submitted by the owner/operator to the regulatory agency and has subsequently been reviewed and approved. The Certificate of Approval to Operate is issued after a period of 6 months of continuous and successful operation.

1.8.2 Operator Requirements

Refer to Appendix A.

1.9 MONITORING REQUIREMENTS

A monitoring program, including regular sampling and analysis of sewage treatment plant effluent and recording of flows, shall be undertaken by the systems operating authority/owner. In the case of Newfoundland and Labrador the Department of Environment and Conservation undertakes regulatory monitoring and will establish monitoring requirements in the Permit to Operate. The monitoring program should be carried out in compliance with sampling and analysis requirements set by the appropriate regulatory agency. In the case of Prince Edward Island the frequency of sampling and parameters to be tested are prescribed in legislation.

For monitoring and sampling requirements refer to the Regulatory Agency having jurisdiction.

Samples should be 24-hour composite samples, except for those collected from lagoons and those samples collected for bacteriological testing, which may be grab samples. Samples shall be analyzed for BOD_5 and suspended solids. Additional monitoring parameters shall be listed in the "Approval/Permit to Operate".

1.9.1 Owner/Operator Responsibility

The owner/operator of any wastewater treatment or collection facility shall be responsible for conducting all process control and compliance monitoring. The owner/operator shall ensure that all compliance monitoring is conducted in accordance with Section 1.9 and the stipulations of the facility's "Approval/Permit to Operate".

1.9.2 Regulatory Agencies' Responsibility

The regulatory agency shall be responsible for enforcing compliance requirements, as described in the "Approval/Permit to Operate" issued to any wastewater treatment or collection facility.

1.10 COMPLIANCE REQUIREMENTS

Compliance requirements will be established by regulatory agencies having jurisdiction.

1.11 REPORTING REQUIREMENTS

The operator/authority/owner shall ensure that all monitoring results are submitted to the appropriate regulatory agency in a timely manner or as a minimum as required in the “Approvals/Permit to Operate”.

2.1 TYPE OF SEWERAGE SYSTEM

In general and except for special reasons, the Minister will approve plans for new systems or extensions only when designed upon a separate sewer basis, in which rain water from roofs, streets and other areas and groundwater from foundation drains are excluded. Overflows from intercepting sewers should not be permitted at points where they will adversely affect a watercourse or the use of water therefrom. Otherwise provision shall be made for treating the overflow.

2.2 DESIGN CAPACITY CONSIDERATIONS

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

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

a. maximum hourly domestic sewage flow;

b. additional maximum sewage or waste from industrial plants;

c. inflow and groundwater infiltration;

d. topography of area;

e. location of waste treatment plant;

f. depth of excavation; and

g. pumping requirements.

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

2.3 HYDRAULIC DESIGN

2.3.1 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 entering the sewage system.

2.3.2 Extraneous Sewage Flows

2.3.2.1 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. The connection of foundation drains to sanitary sewers is not recommended. 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. It is recommended that foundation drainage be directed either to the surface of the ground or into a storm sewer system, if one exists.

2.3.2.2 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.

2.3.2.3 Extraneous Flow Allowances

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.

a) General Inflow/Infiltration Allowance

A general inflow/infiltration allowance based on either area or length and diameter of pipe 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.

- The area allowance ranges from 0.14 to 0.28 l/sec per gross hectare.
- The length and diameter of pipe allowance ranges from 0.24 to 0.48 m^3/cm of pipe diameter/km length of pipe/day.

b) 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/sec 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.

2.3.3 Domestic Sewage Flows

Unless actual flow measurement has been conducted, the following criteria should be used in determining peak sewage flows from residential areas, including single and multiple housing, mobile home parks, etc.:

a. design population derived from drainage area and expected maximum population over the design period;

b. average daily domestic flow (exclusive of extraneous flows) of 340 l/cap·d;

c. peak extraneous flow (including peak infiltration and peak inflow); and

d. peak domestic sewage flows to be calculated by the following equation:

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

where:

Q(d) = peak domestic sewage flow (including extraneous flow) in l/sec.

P = design population, in thousands

q = average daily per capita domestic flow in l/cap·d. (exclusive of extraneous flows)

M = peaking factor (as derived from

Harman Formula **Babbit Formula**

$$M = 1 + \frac{14}{4 + P^{0.5}} \quad \text{or} \quad M = \frac{5}{P^{0.2}} \quad \text{or as}$$

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

I = unit of peak extraneous flow, in l/sec per hectare.

A = tributary area in gross hectares.

i	=	unit of peak extraneous flow, in m^3/cm of pipe diameter/km length of pipe/day.
D	=	diameter of pipe in cm.
L	=	length of pipe in km.
S	=	unit of manhole inflow allowance for each manhole in sag location, in l/sec.
N	=	number of manholes in sag locations.

2.3.4 Commercial and Institutional Sewage Flows

2.3.4.1 Flow Variation

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

2.3.4.2 Flow Equivalent

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

2.3.4.3 Individual Flow Rate

For individual commercial and institutional users the sewage flow rates in Table 2.1 are commonly used for design.

TABLE 2.1 - SEWAGE FLOWS (AVERAGE DAILY)		
Type of Establishment		**(L /day)**
Residence	Private Dwelling	340 per person
	Apartment Building	340 per person
Transient Dwelling units	Hotels	340 per bedroom
	Lodging Houses and Tourists homes	270 per bedroom
	Motels and Tourist Cabins	300 per bedroom(add for restaurant)
Industrial and Commercial Buildings	(does not include process water or cafeteria)	45 per employee
	(with showers)	90 per employee
Camps	Campsite	500 per campsite
	Trailer Camps (Private Bath)	340 per person
	Trailer Camp (Central Bath, etc)	230 per person
	Trailer Camp (Central Bath, Laundry)	300 per person
	Luxury Camps (Private Bath)	340 per person
	Children's Camps (Central Bath, etc)	230 per person
	Labour Camps	225 per person
	Day Camps - No meals	70 per person
Restaurants (including washrooms)	Average Type(2 x Fire Commissioners capacity)	225 per seat + 100 per employee
	Bar/Cocktail Lounge (2 x Fire Commissioners capacity)	25 per patron
	Short order or Drive-In Service	25 per patron
	24 hour	225 per seat
	Non 24 hour	160 per seat
Clubhouses	Residential Type	340 per person
	Non-Residential Type (Serving Meals)	160 per person
	Golf Club	40 per member
	Golf Club (with bar and restaurant add)	115 Seat
Institutions	Hospitals	950 per bed
	Other Institutions	450 per resident
Schools	Basic	50 per person
	With cafeteria	70 per person
	With Cafeteria and Showers	90 per person
	With Cafeteria, Showers and Laboratories	115 per person
	Boarding	340 per person
Theatres	Theatre (Indoor)	25 per seat
	Theatre (Drive-In With Food Stand)	25 per car
Automobile Service Stations	No Car Washing	20 per car served
	Car Washing	340 per car washed

TABLE 2.1 - SEWAGE FLOWS (AVERAGE DAILY) continued		
Type of Establishment		**(L /day)**
Miscellaneous	Stores, Shopping Centres & Office Buildings	6 per m^2
	Factories (8-hour shift)	115 per person
	Self-service Laundries	1800 per machine
	Bowling Alleys	900 per alley
	Swimming Pools and Beaches	70 per person
	Picnic Parks (With Flush Toilets)	50 per person
	Fairgrounds (based upon average attendance)	25 per person
	Assembly Halls	35 per seat
	Airports (Based on passenger use)	15 per passenger
	Churches	25 per seat
	with Kitchen	35 per seat
	Beauty Parlours	200 per seat
	Barber Shops	75 per seat
	Hockey Rinks	15 per seat
	Day Care Centre	115 per child
	Liquor Licence Establishments	115 per seat
	Mobile Home Parks	1350 per space
	Nursing and Rest Homes	450 per resident
	Senior Citizen Home	600 per apartment
	Recreational Vehicle Park	180 per space

2.3.4.4 *Peak Factor*

When using the above unit demands, maximum day and peak rate factors must be developed. For establishments in operation for only a portion of the day, such as schools, shopping plazas, etc., the water usage should also be factored accordingly. For instance, with schools operating for 8 hours per day, the water usage rate 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.

2.3.5 Industrial Sewage Flows

2.3.5.1 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.

2.3.5.2 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 the overall allowances are not exceeded. Industries with the potential to discharge sewage at higher than the accepted rate could either be barred from the area, or be required to provide flow equalization and/or off-peak discharge facilities, or be restricted by a sewer-use by-law.

2.3.5.3 Flow Allowances

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

2.3.6 Combined Sewer Interceptors

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

2.3.6.1 Combined Sewer Overflows[1]

Combined sewer systems (CSSs) are wastewater collection systems that transport both sanitary sewage and stormwater in a single pipe to a treatment facility. During periods of heavy rainfall or wet weather the capacity of the CSS and/or treatment facility may be exceeded resulting in direct discharges of untreated wastewater to receiving environments. These overflows are referred to as combined sewer overflows (CSOs). Requirements for CSO treatment shall be as specified by the regulatory agency having jurisdiction.

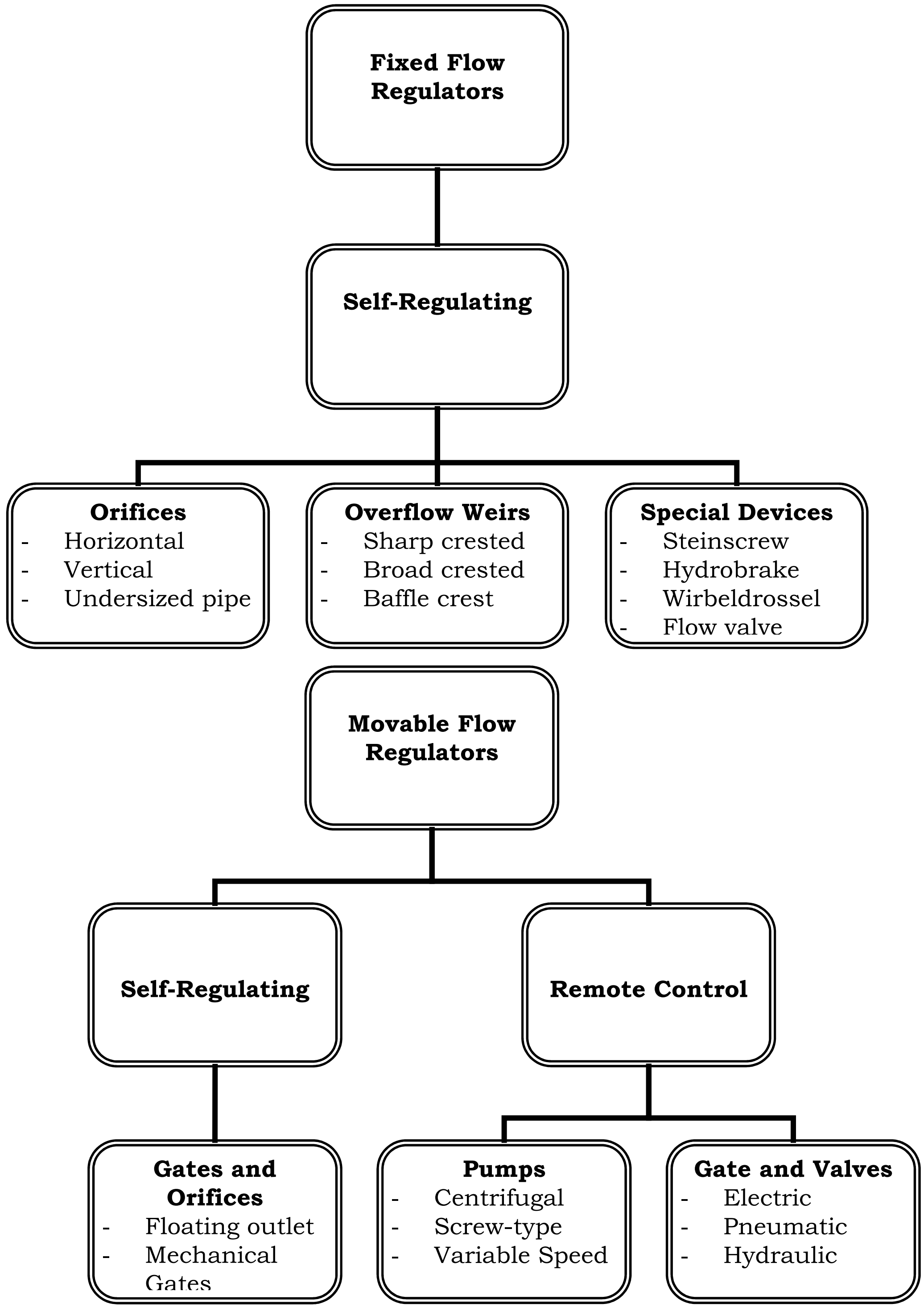
The design requirements for sanitary sewers outlined in this manual specify that all new sewer systems be designed as separate sewers. There will, however, still remain many existing combined sewer systems. This will result in the continued existence of CSOs. This being the case, all receiving water quality studies and waste load allocation models must take into account the effect of CSOs. It is the objective of the regulatory agencies to reduce, where possible and practical, the frequency and duration of CSOs so as to minimize their associated impacts on nearby receiving water.

Maximization of storage is a measure used to reduce the magnitude, frequency and impact of CSOs without significant construction or expense. In order to maximize in-line storage in the collection system, control measures downstream of the excess capacity typically are used. These include the following:

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

The figure below classifies some of the various types of regulating structures for outlet control.

FIGURE 2.1
VARIOUS TYPES OF OUTFLOW CONTROL DEVICES



2.3.6.1.1 CSO Control Methods[2]

I. Source Controls (Best Management Practices)

1. Porous pavements
2. Flow detention
3. Rooftop storage
4. Area drain and roof leader disconnection
5. Utilization of pervious areas for recharge
6. Air pollution reduction
7. Solid waste management
8. Street sweeping
9. Fertilizer and pesticide control
10. Snow removal and de-icing control
11. Soil erosion control
12. Commercial/Industrial runoff control
13. Animal waste removal
14. Sewer line flushing
15. Catch basin cleaning
16. Identifying and/or eliminating sewer system cross connections
17. Public Education programs

II. Collection System Controls

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

III. Storage

1. In-system storage
   a. Inflatable dams
   b. Manual and automatic valves and gates
2. Surface storage
3. Off-line storage
   a. Storage tanks
   b. Lagoons
   c. Deep tunnels
   d. Abandoned pipelines
   e. In-receiving water flow balance method
   f. Street storage

IV. Physical Treatment
1. Sedimentation
2. Dissolved air flotation
3. Screens
    a. Bar screens and coarse screens
    b. Fine screens and microstrainers
4. Filtration
5. Flow concentrators

V. Biological Treatment
1. Activated sludge
2. Trickling filtration
3. Rotating biological contractors
4. Treatment lagoons
    a. Oxidation ponds
    b. Aerated lagoons
    c. Facultative lagoons
5. Land treatment

VI. Physical-Chemical Treatment
1. Chemical clarification
2. Filtration
3. Carbon absorption
4. High gradient magnetic separation

VII. Chemical Treatment (disinfection)
1. Chemical
2. Radiation

2.3.6.1.2 Treatment for Combined Sewer Overflows[2]

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

a) Physical

Physical treatments alternatives include sedimentation, dissolved air floatation, screening and filtration. Physical treatment operations are usually flexible enough to be readily automated and can operate over a wide range of flows. Also, they can stand idle for long period of times without affecting treatment efficiencies.

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

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

b) Biological and Physical-Chemical Treatment

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

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

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

c) Chemical Treatment (Disinfection)

Refer to Chapter 8 for disinfection requirements.

2.4 DETAILS OF DESIGN AND CONSTRUCTION

2.4.1 Sewer Capacity

Sewers shall be designed to handle the peak anticipated sewage flow when flowing full.

2.4.2 Pressure Pipes

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

2.4.3 Minimum Pipe Size

No public sewer shall be less than 200 mm in diameter. However, under limited circumstances, such as effluent from Septic Tank Effluent Pump Systems (STEP) and Septic Tank Effluent Gravity Systems (STEG), sewers of not less than 100-mm diameter may be allowed if the owner can demonstrate that the proposed sewer size is adequate and will not be detrimental to the operation and maintenance of the sewer system.

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

For small diameter low pressure or vacuum sewer collection systems, the designer shall provide hydraulic calculations and/or supporting information to verify the proposal.

2.4.4 Depth

In general, sewers shall be deep enough to prevent freezing and to receive sewage from most basements.

Insulation shall be provided for sewers that cannot be placed at a depth sufficient to prevent freezing.

2.4.5 Slope

Sewers shall be laid with a uniform slope between manholes with the exception of alternate wastewater collection systems.

2.4.5.1 Minimum Slopes

All sewers shall normally be designed and constructed to give mean velocities, when flowing full, of not less than 0.6 metres per second or greater than 4.5 metres per second based on Kutter's or Manning's formula using "n" value of 0.013. Use of other practical "n" values may be permitted by the reviewing agency if deemed justifiable. Velocities above 4.5 m/s may be permitted with high velocity protection. The following are the minimum slopes which will provide a velocity of 0.6 m/s when sewers are flowing full:

TABLE 2.2 - MINIMUM SLOPES FOR FULL-PIPE VELOCITY OF 0.6 M/S

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

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

2.4.5.2 *Increased Slopes*

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

2.4.5.3 *Reduced Slopes*

Under special conditions, if full and justifiable reasons are given, slopes slightly less than those required for the 0.6 metre per second 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 furnish with his report his 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 operating authority of the sewer system will give written assurance to the appropriate reviewing agency that any additional sewer maintenance required by reduced slopes will be provided.

2.4.5.4 *High Velocity Protection*

Where velocities greater than 4.5 metres per second are unavoidable, special provisions shall be made to protect against displacement by erosion and shock.

2.4.5.5 Steep Slope Protection

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

a. not over 11 metres centre to centre on grades 20 percent and up to 35 percent.

b. not over 7.3 metres centre to centre on grades 35 percent and up to 50 percent.

c. not over 5 metres centre to centre on grades 50 percent and over.

2.4.6 Alignment

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

2.4.7 Curvilinear Sewers

Curvilinear sewers may be considered for pipe sizes in excess of 600 mm with the following restrictions applicable:

1. The sewer shall be laid as a simple curve of a radius equal to or greater than 60 m.

2. Manholes shall be located at the ends of the curve and at intervals not greater than 90 m along the curve.

3. The curve shall run parallel to the curb or street centre line.

4. The minimum grade on curved sewers shall be fifty percent greater than the minimum grade required for straight runs of sewers. This requirement will be waived if the designer submits calculations to demonstrate that increased slope is not required to achieve self-cleansing velocity.

5. Length of pipe shall be such that deflections at each joint shall be less than the allowable maximum recommended by the manufacturer.

6. In general, curved sewers should be used only where savings in costs or the difficulty of avoiding other utilities necessitates their use.

2.4.8 Changes in Pipe Size

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

2.4.9 Allowance for Hydraulic Losses at Sewer Manholes

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

Head loss Across Manholes

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

where:

H	=	Head loss	m
k	=	coefficient	dimensionless
V_1	=	entrance velocity	m/s
V_2	=	exit velocity	m/s
g	=	acceleration due to gravity	m/s^2

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

Table 2.3 – Recommended Invert Drop

Invert Drop	
a) straight run	15 mm
b) 45 degree turn	30 mm
c) 90 degree turn	60 mm

2.4.10 Sewer Services

Sewer services shall be consistent with the Local Municipality Authority or Provincial Plumbing and Drainage Regulations. It is required that unless Tees or "Wyes" have been installed, that saddles be used in connecting the service to the sewer. Generally these are placed at an angle of 45 degrees above horizontal. Connections shall be made by authorized personnel only.

Pipes with watertight and root proof joints should be used for house connections. Minimum pipe size should be 150 mm diameter for double connections and 100 mm diameter for single connections.

2.4.11 Sulphide Generation

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

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

2.4.12 Materials

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

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

2.4.13 Metering and Sampling

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

2.4.14 Sewer Extensions

In general, sewer extensions shall be allowed only if the receiving sewage treatment plant is either:

a. Capable of adequately processing the added hydraulic and organic load or

b. Provision of adequate treatment facilities on a time schedule acceptable to the approving agencies is assured.

2.4.15 Installation

2.4.15.1 Standards

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

2.4.15.2 Trenching

a. The width of the trench shall be ample to allow the pipe to be laid and jointed properly and to allow the backfill to be placed and compacted as needed. The trench sides shall be kept as nearly vertical as possible.

When wider trenches are dug, appropriate bedding class and pipe strength shall be used.

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

2.4.15.3 Foundation

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

2.4.15.4 Bedding

The sewer pipe should be bedded on carefully compacted granular material. The granular material shall have a minimum thickness of 150 mm and cover the full width of the trench.

In general, a well-graded crushed stone is a more suitable material for sewer pipe bedding than a uniformly graded pea gravel. For small sewer pipes, the maximum size should be limited to about 10% of the pipe diameter. Crushed stone or gravel meeting the requirement of ASTM Designation C33, Gradation 67 (19-9.8 mm) will provide the most satisfactory sewer pipe bedding. However, the recommendation of the manufacturer should also be taken into consideration when specifying a particular bedding material. Material removed from the trench shall not be used as bedding material.

2.4.15.5 Haunching

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

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

2.4.15.6 Initial Backfill

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

2.4.15.7 Final Backfill

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

The material consists of the excavated material containing no organic matter or rocks having any dimension greater than 200 mm. In most cases, final backfill does not affect the pipe design. Compaction of the final backfill is usually controlled by the location as follows: traffic areas; 95% of modified Proctor density required; general urban areas; 90% of modified Proctor density may be adequate; undeveloped areas; 85% of modified Proctor density may be required. Trench backfilling should be done in such a way as to prevent dropping of material directly on the top of pipe through any great vertical distance.

2.4.15.8 Borrow Materials

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

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

2.4.15.9 Deflection Test

a. Deflection test shall be performed on all flexible pipe. The test shall be conducted after the final backfill has been in place at least 30 days to permit stabilization of the soil-pipe system.

b. No pipe shall exceed a deflection of 5 percent. If deflection exceeds 5 percent, replacement or correction shall be accomplished in accordance with requirements in the approved specifications.

c. The rigid ball, mandrel or an approved electronic device used for the deflection test shall have a diameter not less than 95% of the base inside diameter or average inside diameter of the pipe depending on which is specified in the ASTM Specification, including the appendix, to which the pipe in manufactured. The test shall be performed without mechanical pulling devices.

2.4.16 Joints

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

2.4.17 Sewer Rehabilitation Methods[3]

2.4.17.1 Sewer Replacement

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

2.4.17.2 Sewer Relining

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

2.4.17.2.1 Lining and Slip lining

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

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

2.4.17.2.2 Inversion Lining

Because it has close contact with the inside of the original pipe, inversion lining eliminates annular space leakage. If the part of the lining that covers the laterals is cut out properly, leakage around the laterals can be reduced to a low value. Lack of care in this step can result in poor infiltration control. Inversion lining can be effective in controlling I/I as a replacement method and does not require excavation to reconnect laterals if the existing lateral to main line hookup is in sound condition.

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

2.4.17.3 Sewer Sealing

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

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

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

2.4.17.4 Service Lateral Rehabilitation

Service laterals can constitute a serious source of both infiltration and inflow. They can contribute up to 75% or more of peak infiltration flows. The rehabilitation methods applied to the main sewer line, including slip lining, inversion lining, and grouting have been adapted for rehabilitating service laterals in addition to excavation and replacement.

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

2.4.17.5 Inflow Control

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

2.4.17.5.1 Manholes

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

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

2.4.17.5.2 Catch Basins

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

2.4.17.5.3 Roof drain

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

2.4.17.5.4 Other

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

2.4.18 Directional Drilling[3]

This technique is mainly used for the installation of long, vertically curved pipelines, usually under bodies of water such as rivers, estuaries, and canals. Using substantial surface equipment and being capable of drives to more than 1000 m, the technique is best suited to major schemes that need expensive and heavy equipment. Directional drilling can also be used for service connections. In this technique, a small-diameter pilot hole is drilled in a shallow arc. A washover pipe slightly larger than the pilot tube follows the drill string, acting both as temporary support and a method of reducing friction on the drill string before enlargement. The completed pilot bore is enlarged using backreaming techniques until large enough to receive the final pipe, which is normally steel, although polyethylene and bundles of pipes also have been used.

2.5 MANHOLES

2.5.1 Location

Manholes shall be located at all junctions, changes in grade, size or alignment (except with curvilinear sewers) and termination points of sewers.1

2.5.2 Spacing

2.5.2.1 Normal Spacing

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

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

2.5.3 Minimum Diameter

The minimum diameter of a sanitary manhole shall be 1050 mm.

2.5.4 Drop Manholes

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

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

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

2.5.5 Manhole Bases

Precast bases may be used for manholes up to 9 m deep.

2.5.6 Pipe Connections

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

2.5.7 Frost Lugs

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

2.5.8 Frame and Cover

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

a. adequate strength to support superimposed loads;
b. provision of a good fit between cover and frame to eliminate movement in traffic; and
c. a reasonably tight closure.

2.5.9 Watertightness

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

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

2.5.10 Flow Channel and Benching

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

2.5.10.1 Small Pipe Channel

For sewer sizes less than 375 mm, the channel height should be at least one half the pipe diameter.

2.5.10.2 Large Pipe Channel

For sewer sizes 375 mm and larger, the channel height should not be less than three-fourths of the pipe diameter.

2.5.10.3 Bench Area

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

2.5.10.4 Bench Slope

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

2.5.11 Corrosion Protection

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

2.6 TESTING AND INSPECTION

2.6.1 General

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 it's related appurtenances, shall be flushed prior to testing. The method of testing shall be as described in the construction specifications. In the absence of such specifications the following testing method will apply.

2.6.2 Exfiltration Test

Each sewer section shall be filled with water and a nominal head shall remain on the section for twenty-four hours immediately prior to testing.

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

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

2.6.3 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.

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

2.6.4 Allowable Leakage

Allowable leakage shall be determined by the following formula:

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

where:

L = allowable leakage in litres per hour
D = diameter in mm
S = Length of section, in metres
F = leakage factor, (litres per hour per mm of diameter per 100 metres of sewer):

Exfiltration Test:

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

Infiltration Test:

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

2.6.5 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 two minute period the air supply shall be shut off, and the pressure decreased to 24 kPa. The time required for the pressure to reach 20.55 kPa shall be measured.

2.6.6 Allowable Time for Air Pressure Decrease

Minimum times allowed for air pressure drop are provided in the following table:

TABLE 2.4 - MINIMUM SPECIFIED TIME REQUIRED FOR A 3.45 kPa PRESSURE DROP FOR SIZE AND LENGTH OF PIPE INDICATED FOR Q = 0.000457 $m^3/min/m^2$ OF INTERNAL SURFACE

Pipe Dia. (mm)	Min. Time (min: sec)	Length for Min. Time (m)	Time for Longer Length (sec)	Specification Time for Length (L) Shown (min:sec)							
				30 m	45 m	60 m	75 m	90 m	105 m	120 m	135 m
100	1:53	182	0.190L	1:53	1:53	1:53	1:53	1:53	1:53	1:53	1:53
150	2:50	121	0.427L	2:50	2:50	2:50	2:50	2:50	2:50	2:51	2:51
200	3:47	91	0.760L	3:47	3:47	3:47	3:47	3:48	4:26	5:04	5:42
250	4:43	73	1.187L	4:43	4:43	4:43	4:57	5:56	6:55	7:54	8:54
300	5:40	61	1.709L	5:40	5:40	5:42	7:08	8:33	9:58	11:24	12:50
375	7:05	48	2.671L	7:05	7:05	8:54	11:08	13:21	15:35	17:48	20:02
450	8:30	41	3.846L	8:30	9:37	12:49	16:01	19:14	22:26	25:38	28:51
525	9:55	35	5.235L	9:55	13:05	17:27	21:49	26:11	30:32	34:54	39:16
600	11:20	30	6.837L	11:24	17:57	22:48	28:30	34:11	39:53	45:35	51:17
675	12:45	27	8.653L	14:25	21:38	28:51	36:04	43:16	50:30	57:42	46:54
750	14:10	24	10.683L	17:48	26:43	35:37	44:31	53:25	62:19	71:13	80:07
825	15:35	22	12.926L	21:33	32:19	43:56	53:52	64:38	75:24	86:10	96:57
900	17:00	20	15.384L	25:39	38:28	51:17	64:06	76:55	89:44	102:34	115:23

2.6.7 Sewer Inspection

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

2.6.7.1 Video Inspection

Inspection on 100% of the sewer using the closed circuit television method and recorded on videotape should be specified. This should be conducted within the one-year guarantee period. This inspection should be carried out preferably during the periods of high ground water table in the spring or fall, or at the discretion of the regulatory agencies.

2.6.7.2 Inspection Record

The complete record of the inspection shall be the property of the owner or the municipality. The original video and one edited copy of the video of the sections showing defects shall be turned over to the owner or municipality.

2.6.7.3 Record Content

The maximum speed of the television camera through the pipe shall be 0.30 metres per second with a 5-second minimum stop at each defective location and a 15 - second minimum stop at each lateral showing a flow discharging into the pipe. The audio part shall include the recording of distances at a maximum interval of three metres and a brief description of every defective location and of each service connection.

2.7 INVERTED SIPHONS

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

2.8 PROTECTION OF WATER SUPPLIES

2.8.1 Water-Sewer Cross Connections

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

2.8.2 Relation to Water Works Structures

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

2.8.3 Relation to Water Mains

2.8.3.1 Horizontal and Vertical Separation

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

a. it is laid in a separate trench, or if;

b. it is laid in the same trench, with the water main located at one side with a minimum horizontal separation of 300 mm and on a bench of undisturbed earth and if;

c. in either case the elevation of the top (crown) of the sewer is at least 300 mm below the bottom (invert) of the water main or as required by the Regulatory Agency having jurisdiction.

d. Where a water main must be installed paralleling a gravity sewer and at a lower elevation than the gravity sewer, the water main must be installed in a separate trench. The soil between the trenches must be undisturbed.

2.8.3.2 Crossings

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

2.8.3.3 Special Conditions

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

2.8.3.4 Warning/Marker and Detection Tape

Warning/marker and detection tape should 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.

2.9 SEWERS IN RELATION TO STREAMS

2.9.1 Location of Sewers on Streams

2.9.1.1 Cover Depth

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

a. 0.3 m of cover is required where the sewer is located in rock;

b. 0.9 m of cover is required in other material. In major streams, more than 0.9 m of cover may be required.

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

Less cover will be approved only if the proposed sewer crossing will not interfere with the future improvements to the stream channel. Reasons for requesting less cover should be given in the project proposal.

2.9.1.2 Horizontal Location

Sewers located along streams shall be located outside of the stream bed and sufficiently remote therefrom to provide for future possible stream widening and to prevent pollution by siltation during construction.

2.9.1.3 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.

2.9.1.4 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.

2.9.2 Construction

2.9.2.1 Materials

Sewers entering or crossing streams shall be constructed of cast or ductile iron pipe with mechanical joints; otherwise they shall be constructed so they will remain watertight and free from changes in alignment or grade. Material used to backfill the trench shall be stone, coarse aggregate, washed gravel or other materials which will not cause siltation.

2.9.2.2 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 to provide adequate control of siltation and erosion. 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.

2.10 AERIAL CROSSINGS

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

Precautions against freezing, such as insulation and increased slopes shall be provided. Expansion jointing shall be provided between above-ground and below-ground sewers.

For aerial stream crossings the impact of flood waters and debris shall be considered. The bottom of the pipe shall be placed no lower than the elevation of the fifty (50) year flood.

2.11 ALTERNATIVE WASTEWATER COLLECTION SYSTEMS

2.11.1 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 planning a project.

2.11.1.1 Population Density

Conventional sewers are typically costly on a lineal foot 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 foot basis, so often prove to be more cost-effective when serving sparse populations.

2.11.1.2 Ground Slopes

Where the ground profile over the sewer 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-prohibitively deep. 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.

2.11.1.3 Subsurface Obstacles

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

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

2.11.2 Pressure Sewer Systems

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

Each home connected to the pipeline requires either a grinder pump or a septic tank effluent pump (STEP). The major difference between the two pressure systems is in the onsite equipment and layout. Modification of household pumping is not required for either system. Pressure systems do not have the large excess capacity typical of conventional gravity sewers therefore they must be designed with a balanced approach with consideration of future growth and internal hydraulic performance.

2.11.2.1 Grinder Pump System[4]

A Grinder Pump pressure sewer has a pump and electrical service at each service connection. The pumps discharge into a pressurized pipe system that terminates at a treatment facility or gravity collector. Since the mains are pressurized there is no infiltration into them; however, infiltration and inflow can occur in the house sewers and the pump wells. In areas where the Grinder Pump sewer system has replaced septic tank and leaching field system, these may be retained for emergency overflow. They should be separated from the pump well by a gate valve that is opened only when necessary to accommodate emergency overflow from the Grinder Pump unit; otherwise, the septic tank and leaching field can become sources of large volumes of infiltration.

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

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

2.11.2.2 Septic Tank Effluent Pump (STEP) System[4]

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

2.11.2.3 Design Criteria[4]

When positive displacement Grinder Pump systems are used, the design flow can be obtained by multiplying the pump discharge by the maximum number of pumps expected to be operating simultaneously. The following equation is used for centrifugal pumps:

$$Q = 1.262 + 0.032D$$

Where:

Q = flow in l/sec
D = number of equivalent dwelling units served.
*above equation for 20 usgpm pump

The operation of the system under various assumed conditions should be simulated by a computer as a check on the adequacy of the design. Allowances for infiltration and inflow are not required. No minimum velocity is generally used in design, but Grinder Pump systems must attain 1 – 1.5 m/s at least once per day.

A Hazen-Williams coefficient C = 130 to 150 is suggested for hydraulic analysis.

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

Grinder Pump and STEP pumps are sized to accommodate the hydraulic grade requirements of the system. Air release valves are placed at high points in the sewer and often vented to soil beds. Grinder Pump effluent is generally about twice the strength of the conventional sewer wastewater (e.g. BOD and TSS of 350mg/l). STEP effluent is pre-treated and has a BOD_5 of 100 to 150mg/l and SS of 50 to 70 mg/l.

2.11.2.4 Monitoring[4]

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

2.11.2.5 System Layout

Pressure sewer systems should be laid out taking the following into consideration:

a. Branched layout rather than looped.

b. Maintain cleansing velocities especially when grinder pump type pressure sewers are used.

c. Minimize high head pumping and downhill flow conditions.

d. Locate on lot facilities close to the home for ease of maintenance.

e. Provide for each home to have its own tank and pump.

2.11.3 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.

2.11.3.1 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.

2.11.3.2 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.

2.11.3.3 *Vacuum Station*

Vacuum stations are typically two-storey 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 hr/d 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.

2.11.3.4 *Design Criteria*[4]

There are no universally accepted criteria for vacuum sewers; however the following are commonly used:

The maximum capacity of a 750mm interface valve is 52 l/sec and 1.5m of water is the minimum vacuum head needed to operate an interface valve.

Vacuum sewer system, design rules have been developed largely by studying operating systems. Important design parameters are presented in Table 2.5 and Table 2.6.

TABLE 2.5 – MAIN LINE DESIGN PARAMETERS	
Minimum distance between lifts	6 m
Minimum distance of 0.2 percent slope prior to a series of lifts	15 m
Minimum distance between top of lift and any service lateral	2 m
Minimum slope	0.2%

TABLE 2.6 – GUIDELINES FOR DETERMINING LINE SLOPES[a]	
Line Size	**Use Largest of:**
100 mm Mains	- 0.2% - Ground Slope - 80% of pipe diameter (Between lifts only)
150 mm Mains	- 0.2% - Ground Slope - 40% of pipe diameter (Between lifts only)

a – Assuming minimum cover at top of slope.

Table 2.7 shows at what length the 0.2 percent slope will govern vs. the percentage of pipe diameter for the slopes between lifts.

TABLE 2.7 – GOVERNING DISTANCES FOR SLOPES BETWEEN LIFTS		
Pipe Diameter (mm)	**Distance (m)**	**Governing Factor**
100	<40	80% of pipe diameter
100	>40	0.2% slope
>150	<30	40% of pipe diameter
>150	>30	0.2% slope

The AIRVAC Company has developed Table 2.8 recommending maximum design flows for each pipe size.

TABLE 2.8 – MAXIMUM FLOW FOR VARIOUS PIPE SIZES	
Pipe Diameter (mm)	**Maximum Flow (ℓ/sec)**
100	3.5
150	9.5
200	19.0
250	35.0

The maximum number of homes served for various pipe sizes is presented in the following table.

TABLE 2.9 – MAXIMUM NUMBER OF HOMES SERVED FOR VARIOUS PIPE SIZES	
Pipe Diameter (mm)	**Homes Served**
100	70[a]
150	260
200	570
250	1050

a – The recommended maximum length of any 100mm run is 610 m, which may limit the amount of homes served to a value less than 70.

The sum of friction and lift losses should not exceed approximately 4m of water. Frictional losses may be estimated using a modified Hazen-Williams formula. The recommended height of a lift is 30 cm in 100mm pipe and 46 to 70cm in larger pipes. The loss due to a lift is taken as the invert to invert rise less the internal pipe diameter.

Dual vacuum pumps should each be sized to handle airflow at design conditions. Dual sewage pumps should each be sized to handle design flow. The collection tank volume should be at least three times the working volume. The working volume should be chosen to allow a sewage pump to start every 15 minutes at design flow. A 1,500 l vacuum reserve tank is normally used. 1 to 3 minutes should be the vacuum pump run time.

2.11.3.5 Monitoring[4]

In order to anticipate potential problems the monitoring programs should include cycle counter readings and spot checks of vacuum pressure at various locations in the piping network.

2.11.4 Small Diameter Gravity Sewers

Small diameter gravity sewers (SDGS) require preliminary treatment through the use 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 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.

2.11.4.1 House Connections

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

2.11.4.2 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.

2.11.4.3 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.

2.11.4.4 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.

2.11.4.5 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.

2.11.4.6 Lift Stations

Lift stations are necessary where the elevation differences do not permit gravity flow. Either STEP units (see Section 2.11.2.2) 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.

2.11.4.7 Design Criteria[4]

Peak flows are based on the following formula,

$$Q = 1.262 + 0.032D$$

Where:

Q = flow in l/sec
D = number of equivalent dwelling units served
*above equation for 20 usgpm pump

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

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

Mainline cleanouts are generally spaced at 120 to 300m apart. The septic (interceptor) tank effluent is generally assumed to contain 100 to 150 mg/l BOD_5 and 50 to 75 mg/l SS. Treatment is normally achieved by stabilization pond or by subsurface infiltration.

2.11.4.7.1 Monitoring[4]

Some management schemes involve biannual tank inspection and pumping schedule (e.g. 3 to 5 year for residential users and every year for commercial users). Otherwise, no monitoring plan is typically established.

2.11.5 Detailed Design Guidelines

The above are general design considerations only. For detailed design refer to:

Alternative Sewer Systems, Manual of Practice No. FD-12, Facilities Development, Water Pollution Control Federation, Alexandria, VA, 1986.

U.S. Environmental Protection Agency: Manual: *Alternative Wastewater Collection Systems*, EPA-625/1-91/024, Office of Research and Development, Washington, DC, 1991.

Footnote References

1. Mays, Larry W., "Stormwater Collection Systems Design Handbook", 2001.

2. Metcalf & Eddy Wastewater Engineering, Treatment, Disposal and Reuse, Boston, Massachusetts, 1991

3. Water Environment Federation: Existing Sewer Evaluation & Rehabilitation, Manual of Practice no. FD-6, Alexandria, VA, 1994.

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

Chapter 3 SEWAGE PUMPING STATIONS

3.1 GENERAL

3.1.1 Location

Sewage pumping station structures and electrical and mechanical equipment shall be protected from physical damage from the one hundred (100) year flood. Sewage pumping stations should remain fully operational and accessible during the twenty-five (25) year flood.

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

3.1.2 Design Capacity

3.1.2.1 Separate Sewer Systems

Pumps and controls should be able to pump the expected twenty-five year peak sewage flows, under normal growth conditions, with the largest capacity pump out of operation. Sewage pumping station facilities should be designed to accommodate the expected 25 year peak sewage flows by upgrading pumps and controls. See Section 2.3 for the recommended approach for the calculation of peak sewage flows. In certain cases, where it can be shown that staging of construction will be economically advantageous, lesser design periods may be used provided it can be demonstrated that the required capacity can be "on-line" when needed. Pumping station overflows shall be permitted under the requirements of Section 3.3.

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

3.1.2.2 Combined Sewer Systems

It may be impractical or economical to design a sewage pumping station on a combined sewer system to pump the expected twenty-five year peak sewage flow, with the largest capacity pump out of operation. Under these conditions the following shall be considered in determining the appropriate design capacity:

a) the minimization of combined sewer overflows.

b) the minimization of pumping station overflows as outlined in Section 3.3.

3.1.3 Accessibility

Sewage pumping stations shall be readily accessible by maintenance vehicles during all weather conditions. The facility should be located off the traffic way of streets and alleys.

3.1.4 Grit

Where it may be necessary to pump sewage prior to grit removal, the design of the wet wells should receive special attention and the discharge piping shall be designed to prevent grit settling in pump discharge lines of pumps not operating.

3.1.5 Sewer Entry

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

3.1.6 Fencing

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

3.1.7 Heating

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

3.1.8 Piping System

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

Suction and discharge piping should be sized to accommodate expected peak hourly flows with velocities ranging from 0.8 m/s to 2.0 m/s, where feasible velocities at the low end of the range are preferable. Consideration should be given to providing access ports for such things as sampling, swabbing, and/or flushing discharge pressure gauge(s).

3.1.9 Electrical

All wiring shall be in accordance with the requirements of the Canadian Electrical Code and the local inspection authority.

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

3.1.10 Lighting

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

3.1.11 Safety

The design and construction of all components of wastewater pumping stations shall conform to the safety provisions of the Occupational Health and Safety and Construction Safety Legislation in the region where the pumping station is located.

3.1.12 Construction Materials

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

3.2 DESIGN

3.2.1 Types of Pumping Systems

The type of sewage pumping station should be selected on the basis of such considerations as reliability and serviceability; operation and maintenance factors; relationship to existing stations/equipment; sewage characteristics; flow patterns and discharge; and long-term capital, operating and maintenance costs.

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

3.2.2 Structures

3.2.2.1 Separation

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

3.2.2.2 Equipment Removal

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

3.2.2.3 Access

Suitable and safe means of access shall be provided to dry wells of pump stations and to wet wells or to other parts of the building containing bar screens or mechanical equipment requiring inspection or maintenance. Stairways should be installed, with rest landings not to exceed 3 m vertical intervals.

3.2.3 Pumps and Pneumatic Injectors

3.2.3.1 Duplicate Units

At least two pumps or pneumatic ejectors shall be provided. A minimum of three pumps should be provided for stations handling flows greater than 4500 m^3/d.

3.2.3.2 Protection Against Clogging

The need for and the type of screening facilities required for pumping stations varies with the characteristics of the sewage, size of sewers and the requirements of the operating authority. For wet well/dry well stations, it is generally accepted practice to provide screening in the form of a basket screen or a manually or mechanically cleaned bar screen. Although basket screens may be cumbersome to remove and empty, they have the advantage of not requiring entry of operating staff into the wet well for cleaning operations. With basket screens, guide rails should be tubular and similar to submersible pump guide rails. Manually cleaned bar screens should be provided with 38 mm clear openings in the inclined (60°) and horizontal bars. The vertical sides should be solid. The minimum width should be 600 mm. A drain platform should be provided for screenings. Pumps handling separate sanitary sewage from 750 mm or larger diameter sewers shall be protected by bar screens meeting the above requirements.

3.2.3.3 Pump Openings

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

3.2.3.4 Priming

The pump shall be so placed that under normal operating conditions it will operate under a positive suction head, except as specified in Section 3.2.11.

3.2.3.5 Electrical Equipment

The wet wells of sewage pumping stations may occasionally contain flammable mixtures presenting a potentially hazardous (explosive) environment. As a minimum, electrical installations in these areas should comply with the requirements of the Canadian Electrical Code, Class 1 Zone 2 Hazardous areas, or as otherwise required by the local inspection authority.

3.2.3.6 Intake

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

3.2.3.7 Constant Speed vs. Variable Speed Pumps

In certain instances, such as pumping stations discharging directly into mechanical sewage treatment plants or into other pumping stations, some means of flow pacing may be required. This can be provided by various means, depending upon the degree of flow pacing necessary. If even minor pressure transients caused by pump starting and stopping would have serious effects, solid state, soft start and stop motor starters should be considered. Where flow surges to treatment plants may be detrimental to the treatment process, variable speed control drives should be considered. If minor surges can be tolerated, two-speed pumps or multiple constant speed pumps can be used.

3.2.3.8 Controls

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

Level control devices located in the station wet wells are to be designed as intrinsically safe systems. Float control should be positioned as per Section 3.2.5.5.

3.2.3.9 Alternation

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

3.2.4 Valves

3.2.4.1 Suction Line

Suitable shutoff valves shall be placed on the suction line of each pump except on submersible and vacuum-primed pumps.

3.2.4.2 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. Valves shall be capable of withstanding normal pressure and water hammer.

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

3.2.5 Wet Wells

3.2.5.1 Divided Wells

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

3.2.5.2 Pump Cycle

For any pumping station, the wet well should be of sufficient size to allow for a minimum of a fifteen minute cycle time for each pump. For a two-pump station, the volume of the wet well in cubic metres, between pump start and pump stop should be 0.225 times the pumping rate of one pump, expressed in ℓ/sec. For other numbers of pumps, the required volume of the wet well depends upon the operating mode of the pumping units. Maximum recommended starts per hour are 6 for dry pit motors and 12 for submersible motors.

3.2.5.3 Size

Wet well size and control settings should be based on consideration of the volume required for pump cycling; the design fill time, dimensional requirements to avoid turbulence problems; vertical separation between pump control points; inlet sewer elevation; capacity required between alarm levels and basement flooding and/or overflow elevations; etc. Wet wells should be designed to prevent septicity problems.

3.2.5.4 Floor Slope

The wet well floor shall have a minimum slope of 1 to 1 to the hopper bottom. The horizontal area of the hopper bottom shall be no greater than necessary for proper installation and function of the inlet.

3.2.5.5 Float Controls

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

3.2.5.6 Pump Start Elevation

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 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.

3.2.5.7 Pump Stop Elevation

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

3.2.5.8 Bottom Elevation

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

3.2.5.9 Air Displacement

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

3.2.5.10 Location of Valves

Valves should not be located in the wet well unless permitted by Regulatory Authority having jurisdiction.

3.2.6 Dry Wells

3.2.6.1 Dry Well Dewatering

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

3.2.6.2 Maintenance

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

3.2.7 Ventilation

3.2.7.1 General

Adequate ventilation shall be provided for all pump stations. Where the pump pit is below the ground surface, mechanical ventilation is required, so arranged as to independently ventilate the dry well and the wet well. There shall be no interconnection between the wet well and dry well ventilation systems. Ventilation must avoid dispensing contaminants throughout other parts of the pumping station, and vents shall not open into a building or connect with a building ventilation system.

3.2.7.2 Air Inlets and Outlets

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

3.2.7.3 Electrical Controls

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

3.2.7.4 Fans, Heating, and Dehumidification

The fan wheels shall be fabricated from non-sparking material. Automatic heating and dehumidification equipment shall be provided in all dry wells. The electrical equipment and components shall meet the requirements in Section 3.2.3.5.

3.2.7.5 Wet Wells

Ventilation may be either continuous or intermittent. Ventilation, if continuous, should provide at least 12 complete air changes per hour; if intermittent, at least 30 complete air changes per hour. Fresh air shall be forced into the wet well, by mechanical means, at a point 300 mm above the expected high liquid level. There shall be a provision for automatic blow-by to elsewhere in the well, should the fresh air inlet become submerged.

3.2.7.6 Dry Wells

Ventilation may be either continuous or intermittent. Ventilation, if continuous, should provide at least six complete air changes per hour; if intermittent, at least 30 complete air changes per hour. Ventilation shall be forced into the dry well at a point 150 mm above the pump floor, and allowed to escape through vents in the roof superstructure. A system of two speed ventilation with an initial ventilation rate of 30 changes per hour for 10 minutes and automatic change over to 6 changes per hour may be used to conserve heat.

3.2.8 Flow Measurement

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

3.2.9 Water Supply

There shall be no physical connection between any potable water supply and a sewage pumping station which under any conditions might cause contamination of the potable water supply. If a potable water supply is brought to the station it shall be protected with a suitable backflow prevention device (see Section 4.8.2).

3.2.10 Suction Lift Pumps

3.2.10.1 General

Suction lift pumps shall be of the self-priming or vacuum-priming type and shall meet the applicable requirements of Section 3.2. Suction lift pump stations using dynamic suction lifts exceeding the limits outlined in the following sections may be approved by the appropriate reviewing agency upon submission of factory certification of pump performance and detailed calculations indicating satisfactory performance under the proposed operating conditions. Such detailed calculations must include static suction lift as measured from "lead pump off" elevation to centre line of pump suction, friction and other hydraulic losses of the suction piping, vapour pressure of the liquid, altitude correction, required net positive suction head and a safety factor of at least 1.8 metres.

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

3.2.10.2 Self-Priming Pumps

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.

3.2.10.3 Vacuum-Priming Pumps

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 damage due to sewage. 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.

3.2.11 Submersible Pump Stations

3.2.11.1 General

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

Submersible pump stations shall meet the applicable requirements under Sections 3.2.1 to 3.2.10 except as modified in this section.

3.2.11.2 Construction

Submersible pumps and motors shall be designed specifically for raw sewage use, including totally submerged operation during a portion of each pumping cycle. 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.

3.2.11.3 Pump Removal

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

3.2.11.4 Wet Wells

See section 3.2.5 for the layout of wet wells.

3.2.11.5 Mixing for Wet Wells

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

3.2.11.6 Power Supply

Pump power cables, control and alarm circuits shall be designed to provide strain relief and to allow disconnection from outside the wet well. Cable terminations shall be made outside the wet well in enclosures suitably rated for the ambient environment.

3.2.11.7 Controls

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

3.2.11.8 Power Cables

Pump motor cables shall be designed for flexibility and serviceability under conditions of extra hard usage and shall meet the requirements of the Canadian Electrical Code. The ground fault system shall be used to de-energize the circuit in the event of any failure in the electrical integrity of the cable.

3.2.11.9 Valves

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

3.2.11.10 Ventilation

Gravity ventilation may be acceptable for submersible pump stations with a duty pump capacity under 4.7 l/sec provided that maintenance crews carry suitable portable ventilation equipment when visiting the site. Submersible pump stations greater than 4.7 l/sec should have ventilation for the wet well as specified in 3.2.7.5. For continuous ventilation, to facilitate free movement of air, the wet well may be exhausted at the highest elevation level in the structure.

3.2.12 Cathodic Protection

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

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

3.2.13 Alarm Systems

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

3.3 EMERGENCY OPERATION

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

3.3.1 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:

a. 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

b. an in-place or portable pump, driven by an internal combustion engine meeting the requirements of Section 3.3.3 below, capable of pumping from the wet well to the discharge side of the station.

3.3.2 Overflow

If the avoidance of overflows is not possible, provision shall be made for chlorination of the overflow raw sewage unless waived by the regulatory agencies. The overflow facilities should be alarmed and equipped to indicate frequency and duration of overflows, and designed to permit manual flow measurement. Where the operator is signatory to a Shellfish Conditional Area Management Plan, notification and reporting requirements of the plan shall be met. All overflows should be recorded and reported to the regulatory agencies.

3.3.3 Equipment Requirements

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.

3.3.3.1 Engine Protection

The engine must be protected from operating conditions that would result in damage to equipment. Unless continuous manual supervision is planned, protective equipment shall be capable of shutting down the engine and activating an alarm on site and as provided in Section 3.2.13. 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.

3.3.3.2 Size

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

3.3.3.3 Fuel Type

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

3.3.3.4 Engine Ventilation

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

3.3.3.5 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.

3.3.3.6 Protection of Equipment

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

3.3.4 Engine-Driven Pumping Equipment

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

3.3.4.1 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.

3.3.4.2 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 3.3.4.3.

3.3.4.3 Portable Pumping Equipment

Where part or all of the engine-driven pumping equipment is portable, sufficient storage capacity to allow time for detection of pump station failure and transportation and hookup 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.

3.3.5 Engine-Driven Generating Equipment

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

3.3.5.1 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.

3.3.5.2 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 3.3.4.3

3.3.5.3 Portable Generating Equipment

Where portable generating equipment or manual transfer is provided, sufficient storage capacity to allow time for detection of pump station failure 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.

3.4 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.

3.5 FORCE MAINS

3.5.1 Velocity

At design average flow, a cleansing velocity of at least 0.6 metres per second shall be maintained.

3.5.2 Air Relief Valve and Blowoff

An automatic air relief valve shall be placed at high points in the force main to prevent air locking. Drain or blowoff valves should be provided at all low points in pressure sewers.

3.5.3 Termination

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

3.5.4 Design Pressure

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

3.5.5 Size

Force mains shall be sized to provide sufficient flow velocity, required capacity at the available head and to withstand operating pressures as outlined in Sections 3.5.1 and 3.5.4. In general, force mains shall be a minimum of 100 mm in diameter.

3.5.6 Slope and Depth

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

A forcemain should have a minimum cover of 1.8 m.

3.5.7 Special Construction

Force main construction near watercourses or used for aerial crossing shall meet applicable requirements of Sections 2.9 and 2.10.

3.5.8 Design Friction Losses

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

Unlined iron or steel	-	100
All other	-	120

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

3.5.9 Separation from Water Mains

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

3.5.10 Identification

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

3.6 TESTING

3.6.1 General

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

3.6.2 Leakage Test

The force main shall be filled with water, and a test pressure of 1035 kPa or equal to 1.5 times the working pressure shall be applied, measured at the lowest point in the test section. The pressure shall be maintained by pumping water from a suitable container of known volume. The amount of water used for a period of two hours shall be recorded.

3.6.3 Allowable Leakage

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

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

where:

L = allowable leakage in litres/hour

S = length of pipe in metres

D = nominal diameter of pipe in mm

P = test pressure in kPa

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

Chapter 4 SEWAGE TREATMENT FACILITY

4.1 DEFINITION OF SEWAGE TREATMENT PLANT

"Sewage Treatment Plant (STP)" means a facility for the treatment of sanitary wastewater with a discharge of the treated effluent off the site, or effluent dispersal (subsurface or surface irrigation).

4.2 PERFORMANCE EXPECTATIONS

Treatment, the extent of which will depend upon local conditions, shall be provided in connection with all sewer installations. The engineer should confer with the regulatory agencies before proceeding with the design of sewage infrastructure.

4.2.1 Preliminary Treatment

Coarse screens, bar screens and comminutors are generally provided so as to protect downstream pumps and other equipment from damage caused by the movement of large solids and trash.

Usually, grit chambers remove all particles that have settling velocities (in a quiescent settling column) greater than 1.5 to 3 cm/sec at 20℃. Sand particles of specific gravity 2.65 and size 0.2 mm (retained on a 65-mesh screen) are known to have a settling velocity of approximately 3 cm/sec and grit chambers are designed to remove all sand (and gravel) particles of size greater than 0.2 mm. However, particles of specific gravity lower than 2.65 can also have settling velocity greater than 3 cm/sec when they are of very coarse size, and such solids are also removed in a grit chamber whether they are inorganic or organic. Design engineers have no control over this. The only controlling factor is the settling (or subsiding) velocity of a solid particle and this depends on particle size as well as specific gravity (at a given temperature). Consequently, particles collected even in properly designed and operated grit chambers have been known to have wide ranges of specific gravity, size, shape, character and organic content.

Pre-aeration of wastewater can be used to achieve the following objectives:

a. odour control;

b. grease separation and increased grit removal;

c. prevention of septicity;

d. grit separation;

e. flocculation of solids;

f. maintenance of dissolved oxygen (D.O.) in primary treatment tanks at low flows;

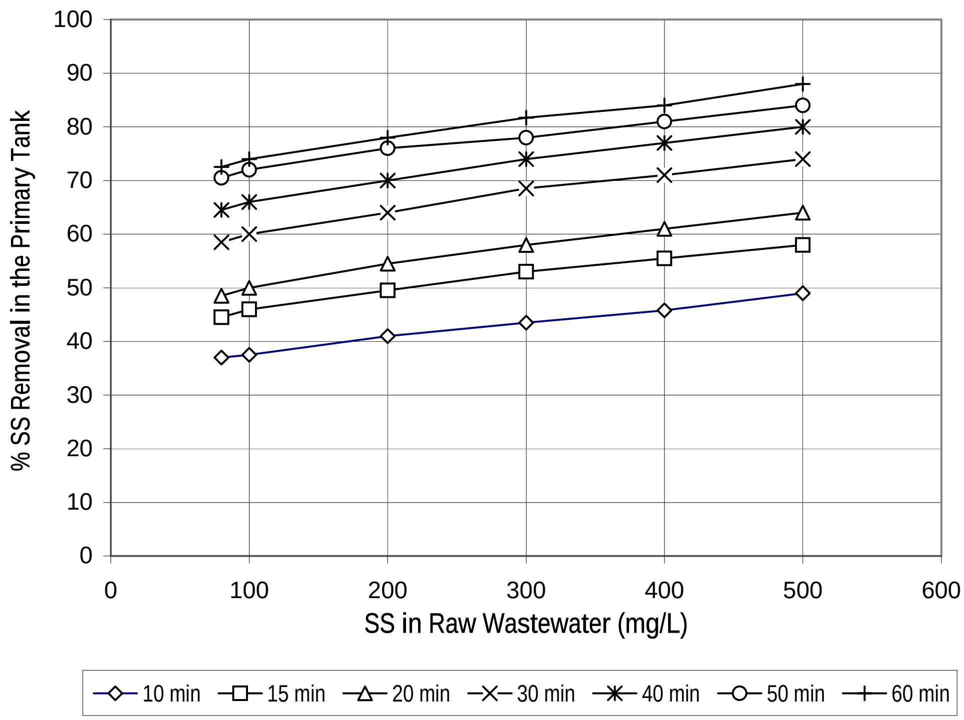
g. increased removal of BOD and SS in primary units (see Figure 4.1); and

h. to minimize solids deposits on side walls and bottom of wetwells.

Pre-chlorination (the practice of applying chlorine at the plant headworks) is used principally to control odour, corrosion and septicity, and to aid in grease removal. The provision to pre-chlorinate influent wastewater should be considered for wastewater treatment plants.

Figure 4.1

Effect of Preaeration on Removal of SS in a Primary Settling Tank



Note: Removal based on a 2 hr retention time in the settling basins

4.2.2 Primary, Secondary and Tertiary Treatment

Table 4.1 lists expected effluent quality produced by well operated treatment facilities treating typical municipal sanitary sewage. The table can be used to illustrate potential effluent quality for selected processes, and as a guide for performance comparisons. Specific facilities may have different treatment objectives and quality requirements. Other treatment processes are available which are not included in Table 4.1. These could be considered on a case by case basis.

TABLE 4.1- SEWAGE TREATMENT PROCESS TYPICAL EFFLUENT QUALITY				
PROCESS	BOD_5 mg/ℓ	TSS mg/ℓ	Total P mg/ℓ	Total N, mg/ℓ
Primary				
(including anaerobic lagoons)	75 -150	50 -110	5 – 7	25 – 45
- With P Removal	45 - 85	25 – 50	1 – 2	20 – 40
Chemically enhanced primary	70-125[f]	105 – 160[f]	8 - 10[f]	Not Available
Secondary				
- Activated Sludge	10 - 25	10 – 25	3.5 – 6.5	15 – 35
- Aerated Lagoons	15 - 30	20 – 35	4 – 7	20 – 40
Facultative Lagoons - Winter to Late Spring - Summer to Late Fall	 25 - 70 10 - 30	 20 – 60 10 – 40	 3.5 – 7 2 – 5	 20 – 35 5 – 10
Advanced				
- Secondary with chemical treatment (P control)	5 - 15	10 - 30	0.5 – 1.5	15 - 35
Other Biological Systems				
Biological Aerated Filters	10 - 20	10 - 20	Not Available	Not Available
Moving Bed Biofilm Reactors	10 - 25	10 – 25	3.5 – 6.5	15 – 35
Membrane Bioreactors	< 5	< 1	< 0.2[a] < 0.5[b]	< 10[c] < 3[d]
Recirculating Sand Filters	< 15	< 15	10 - 30	12 – 30[e]

a – With chemical addition
b – With Bio-P removal
c – With preanoxic zone
d – With preanoxic and postanoxic zones
e – Based on septic tank effluent with 50% nitrogen removal
f – Based on percent removal and the following average raw effluent values from MOP 11 – Operation of Municipal Treatment Plants.
BOD_5 175 mg/ℓ (40-70% removal)
TSS 175 mg/ℓ (60-90% removal)
Total P 11 mg/ℓ (70-90% removal)

4.3 SITE CONSIDERATIONS

4.3.1 Plant Location

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

a. proximity to residential areas;

b. direction of prevailing winds;

c. accessibility;

d. area available for expansion;

e. local zoning requirements;

f. local soil characteristics, geology, hydrology and topography available to minimize pumping;

g. access to receiving stream;

h. downstream uses of the receiving stream; including but not limited to shellfish harvesting areas, public swimming areas, drinking water supply intakes.

i. 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.

j. proximity to surface water supplies and water wells.

k. storm surge;

l. flood protection.

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

4.3.2 Separation Distances

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

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

a. Mechanical plants (including aerated stabilization ponds) shall be located a minimum of 150 m from residences, 30 m from commercial-industrial developments and 30 m from the nearest property lines. Under special circumstances a lesser separation distance to residences may be adopted, provided provision for odour control equipment is provided at the plant; and

b. Waste stabilization ponds shall be located at least 150 m from isolated human habitation and 300 m from built-up areas or as determined by the regulatory agency having jurisdiction.

c. Recirculating sand filters shall be located 30 m from potable water supply wells, 100 m from water supply wells immediately down slope, 3 m from any lot boundary and 9 m down slope of any lot boundary.

d. For infiltration and irrigation separation distances see Sections 10.2.3.6 and 10.3.3.2 respectively.

4.3.3 Flood Protection

The treatment works structures, electrical and mechanical equipment shall be protected from physical damage from the one hundred (100) year flood. Treatment works should remain fully operational and accessible during the twenty-five (25) year flood. This applies to new construction and to existing facilities undergoing major modifications.

4.3.4 General Plant Layout

The general arrangement of the plant within the site should take into account the subsurface conditions and natural grades to provide the necessary facilities at minimum cost.

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

It is not recommended that construction of any of the facilities be in close proximity to a shore line, except where this is unavoidable. Suitable measures must be taken to adequately protect the structures from the effects of wave action and shore erosion.

Within the constraints mentioned above, the designer should work towards a plant layout where the various processing units are arranged in a logical progression to avoid the necessity for major pipelines or conduits to transmit wastewater, sludges, or chemicals from one module to the next, and also to arrange the plant layout to provide for convenience of operation and ease of flow splitting for proposed and future treatment units.

Where site roadways are provided for truck access, the road design should be sufficient to withstand the largest anticipated delivery or disposal vehicles with due allowance for vehicle turning and forward exit from the site.

In order to avoid the dangers of high voltage lines crossing the site, it is suggested that a high voltage pole be located at the property line. Depending on the distance from the terminal pole to the control building, the step-down transformer would be located at the terminal pole or adjacent to the control building. If the distance between the terminal pole and the building is excessive, the transformer should be located adjacent to the building. Then the high voltage connections should be brought by underground cable to the pothead at the transformer. In this way, the primary and secondary terminals of the transformer are fully enclosed and no fence is required around the transformer.

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

4.3.5 Provision For Future Expansion

In addition to the general site considerations outlined in Section 4.3.4 there are a number of allowances needed to provide for economical and practical expansion of the wastewater treatment facilities. Key provisions include:

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

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

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

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

4.4 WATER QUALITY OBJECTIVES AND WATER USE GUIDELINES

The typical level of treatment required for any new treatment plant in the Atlantic Provinces is secondary treatment with disinfection. However, each new plant will be evaluated on a case by case basis. The effluent discharge requirements are outlined in the Sewage Treatment Plant Effluent Discharge Policy which is located in an appendix at the back of this manual. 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.

4.4.1 Waste Assimilation Study Procedures

a) Level of Effort

As part of the pre-design evaluation (described in Section 1.2), the engineer shall determine from the applicable regulatory agencies the level of effort required for the particular waste assimilation study.

The regulatory agencies may conclude that the effects of the proposed project on the receiving water will be minimal. In this case, the regulatory agencies 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 regulatory agencies will determine which parameters will require measurement.

When the regulatory agencies are unsure of the possible effects of a project on the receiving water, they 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 regulatory agencies may, at that point, set effluent limitations. The regulatory agencies 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 regulatory agencies believe that a proposed project may have a significant impact on the receiving water quality. The regulatory agencies will determine RWS data requirements.

b. 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.

4.4.2 Waste Assimilation Capacity

4.4.2.1 General

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

A waste 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 Guidelines 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.

4.4.2.2 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) or as required by the regulatory agency having jurisdiction, and the peak hourly effluent discharge rate (both expressed in the same units).

4.4.2.3 Mixing Zone

4.4.2.3.1 General

It may not be practical to treat all effluents so they meet the Water Quality Objective concentrations. Therefore, some volume of water must be provided for dilution or modification of the waste effluent before the Objectives can be met.

A mixing zone is a region of a waterbody in which an effluent discharge of 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 regulatory authorities to protect beneficial water uses should be achieved. Within the mixing zone, where the objectives are not met, there will be some damage or loss to the aquatic environment. Nevertheless, at no point should conditions be immediately lethal so that swimming organisms cannot evade the area.

4.4.2.3.2 Mixing Zone Requirements

Terms and conditions related to the mixing zones may be outlined in the "Approval/Permit to Operate," based on the minimum requirements outlined below. Inherent in these conditions, a mixing zone may not be used as an alternative to adequate treatment.

1. 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:

   a) in streams and rivers the mixing zone shall be apportioned no more than 25% of the cross-sectional area or volume of flow, nor more than one-third of the river width at any transect in the receiving water during all flow regimes which equal or exceed the 7Q20 flow for the area;

   b) in lakes and other surface impoundments the mixing zone volume shall not exceed 10% of that part of the receiving water available for mixing, and surface water quality objectives must be achieved at all points beyond a 100m radius from the effluent outfall.

2. The mixing zone shall be:

   a) Free from substances in concentrations or combinations which may be harmful to human, animal or aquatic life;

b) free from substances that will settle to form putrescent or otherwise objectionable sludge deposits, or that will adversely affect aquatic life or waterfowl;

c) free from debris, oil, grease, scum or other materials in amounts sufficient to be noticeable in the receiving water;

d) free from colour, turbidity or odour-producing materials that would:

   i) adversely affect aquatic life or waterfowl;
   ii) significantly alter the natural colour of the receiving water;
   iii) directly or through interaction among themselves or with chemicals used in water treatment, result in undesirable taste or odour in treated water, and;

e) 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;

3. The presence of a mixing zone should in no way pose a threat to the species survival of any organism in the receiving water outside the mixing zone.

4. No conditions within the mixing zone should be permitted which:

   a) 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) ; or

   b) cause irreversible responses which could result in detrimental post-exposure effects; or

   c) result in bioconcentration of toxic materials which are harmful to the organism or its consumer; or

   d) attract organisms to the mixing zones, resulting in a prolonged and lethal exposure period.

5. The mixing zone should be designed to 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;

6. Mixing zones should not interfere with the migratory routes, natural movements, survival, reproduction, growth, or increase the vulnerability to predation, of any representative aquatic species, or endangered species;

7. Mixing zones should not interfere with fish spawning and nursery areas;

8. Rapid changes in the water quality which could kill organisms by shock effects must not be present. Such conditions could have the effect of creating a higher toxicity value;

9. Municipal and other water supply intakes and recreational areas, as a general rule, should not lie within a mixing zone. However, knowledge of the effluent characteristics and the type of discharge associated with the mixing zone could allow such a mixture of uses;

10. Mixing zones may overlap unless the combined effects exceed the conditions specified in these mixing zone guidelines;

11. Limitations on mixing zones should be established by the regulatory authorities on a case-by-case basis, where "case" refers to both local considerations and the waterbody as a whole or segments of the waterbody;

12. 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;

13. The design and location of the outfall should be considered on a case-by-case basis to reduce the impact of the mixing zone on the receiving waters;

14. Total loadings into all the mixing zones within a river, lake or segment thereof, must not exceed the acceptable loadings from all point-source discharges required to maintain satisfactory water quality;

15. Mixing zones should not result in contamination of natural sediments so as to cause or contribute to exceedences of the water quality objectives outside the mixing zone.

4.4.3 Waste Assimilation Study Field Procedures

Before any field work is carried out on a stream, river, lake, coastal water, or estuaries the objectives of the study and proposed methodology should be laid out and submitted to the Regulatory Agency having jurisdiction for approval.

4.4.4 Waste Load Allocation Modelling

4.4.4.1 General

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.

4.4.4.2 Model Selection

4.4.4.2.1 General

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 modelled. The most appropriate procedure for selecting a model is, therefore, 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:

a) review of existing data on waste loads, and receiving water quality;

b) preliminary mass balance calculations using simple models or equations that provide analytical solutions for various load sources (combined sewer overflows, nonpoint 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.

4.4.4.2.2 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, with a brief discussion of the considerations involved.

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.

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.

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.

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 members of the support agency, and consult with other experienced professionals.

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 support agency 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 support agency 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.

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 support agency 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 Waste Load Allocation study costs should be consistent with the economic, social, or environmental values associated with the problem and its solution.

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.

SUMMARY:

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 which are technically acceptable. The result of the second step is a list of candidate models which 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.

4.4.4.3 Modelling Procedures

Figure 4.2 outlines the typical development of a site specific water quality model.

4.4.4.3.1 Initial Assessment

The first three steps in Figure 4.2 contribute to the initial assessment activity. The historical data are reviewed and employed in conjunction with initial model runs, which compare calculated and observed water quality to:

- confirm existing or future water quality problems.
- define the loads, sources, and sinks that control water quality.
- define the important reactions that control water quality.
- 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.

4.4.4.3.2 Field Program

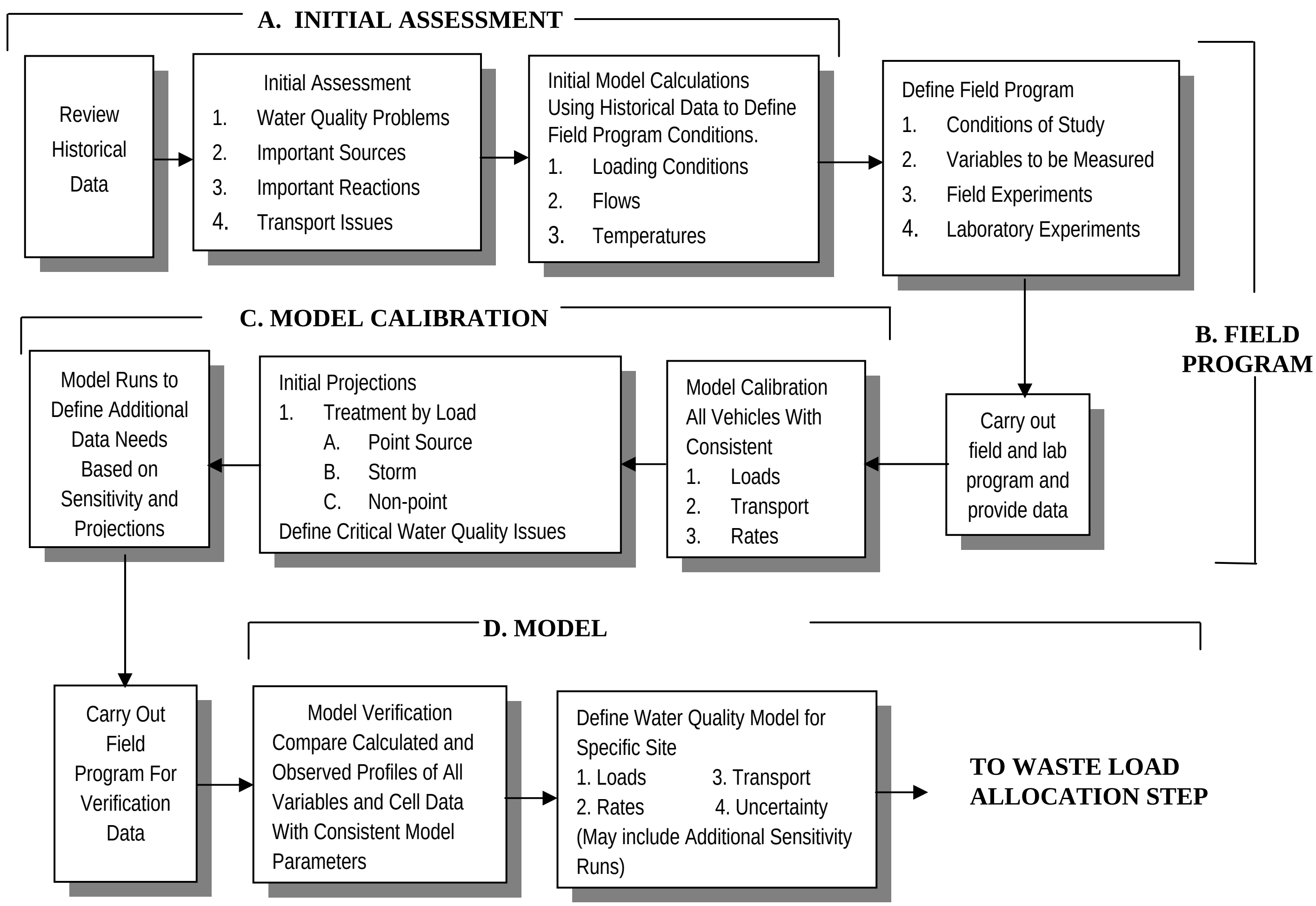
This task translates the results of the initial assessment into a practical field program that can be carried out on the receiving water and in the laboratory using the resources and manpower required and/or available.

4.4.4.3.3 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.

FIGURE 4.2
STEPS IN THE DEVELOPMENT OF A SITE SPECIFIC WATER QUALITY MODEL



4.4.4.3.4 Model Verification

At this stage in the modelling process, a calibrated model has been developed. The next step involves a test of the adequacy of the model in terms of decisions required in the waste load allocation study. Without validation testing, the calibrated model remains a description of the conditions defined by the calibration data set. The uncertainty of any projection or extrapolation of a calibrated model would be unknown unless this is estimated during the validation procedure.

Model verification efforts should contain activities similar to those discussed under model calibration. In general, model parameters should not be altered in the verification analysis. If changes in these model inputs are required, the changes should be entered for all data sets including historical, calibration, and verification data.

Comparison between observed and calculated water quality for all data sets should be developed. Sensitivity analyses should be conducted on model parameters so as to determine which parameters have the greatest impact on model predictions.

The next step in the process is to define a site-specific water quality model that consists of:

- A single set of model parameters that were developed and used in the calibration and verification analysis. These parameters should be uniform in space and time varying only as defined below.

- A set of rules for variation of model parameters in terms of measured information, such as temperature, flow, loads, geometry, etc. The rules for variations of parameters should be those used in the calibration and verification activities.
- A range of values for model parameters that cannot be adequately defined by a single value. The range of parameters, determined from sensitivity analysis, should be used in all projections.

- The quantitative and qualitative measures of model adequacy, including graphs, statistics and appropriate discussions.

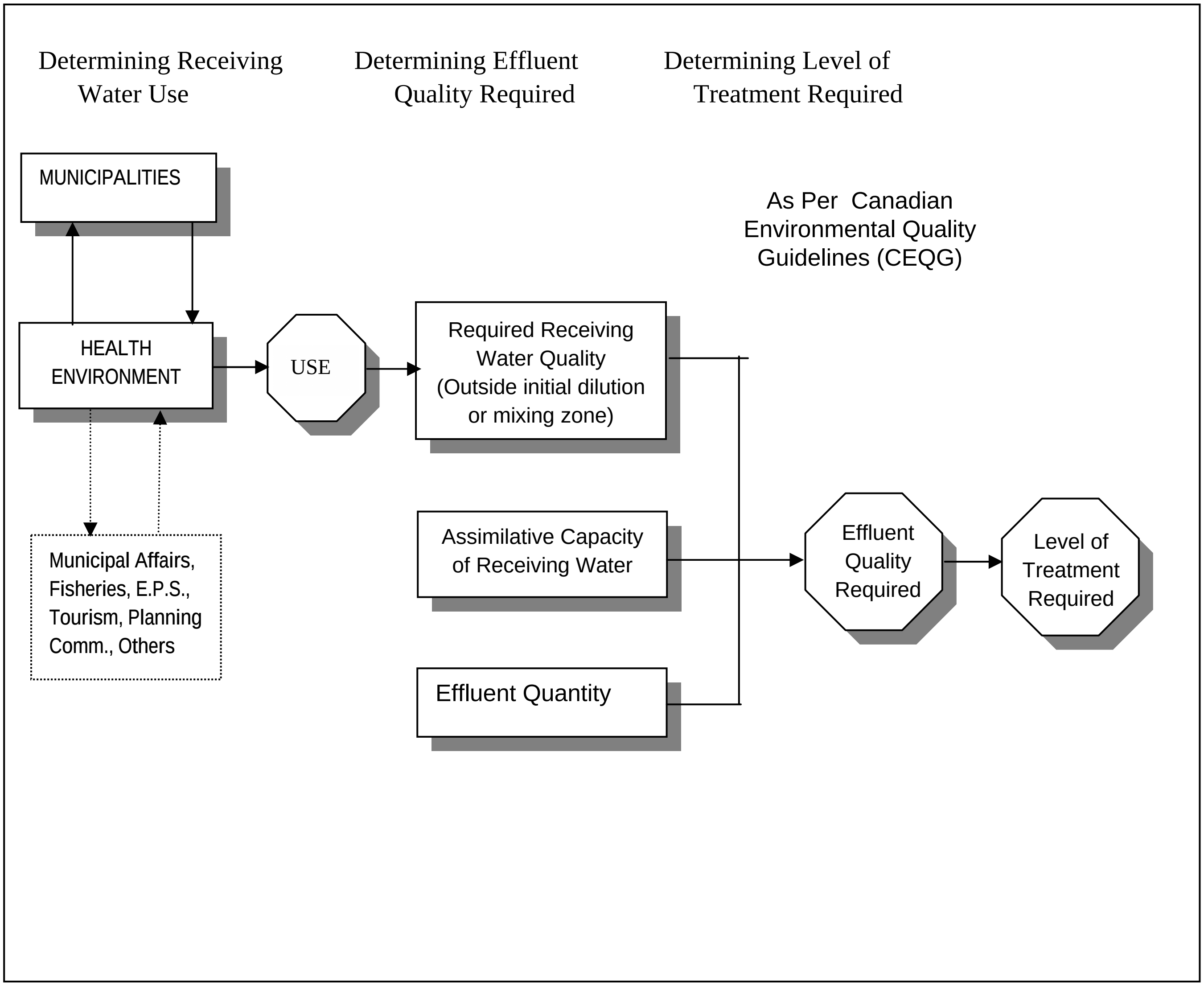
4.4.4.3.5 Allocating Waste Loads

The purpose of the waste load allocation analysis is to define the quantity of waste that may be discharged into a receiving water while meeting the water quality objectives.

The initial requirement is to quantitatively define the critical conditions that will control waste load allocations. There may be one or more critical conditions that should be considered.

Once the critical conditions are established, the calibrated model can be used to predict the water quality response due to various loads. With the model in place, the regulatory agencies shall determine the receiving water use, and thus the required receiving water quality (as outlined in the CCREM guidelines). By varying the loads from the calibrated model, it is then possible to determine required effluent quality and the associated level of treatment required. This procedure is illustrated in Figure 4.3.

FIGURE 4.3
SCHEMATIC REPRESENTATION OF PROCEDURE FOR ESTABLISHING REQUIRED LEVEL OF TREATMENT



4.5 GENERAL DESIGN REQUIREMENTS

4.5.1 Type of Treatment

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

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

a. present and future effluent requirements;

b. location of and local topography of the plant site;

c. space available for future plant construction;

d. the effects of industrial wastes likely to be encountered;

e. ultimate disposal of sludge;

f. system capital costs;

g. system operating and maintenance costs, including basic energy requirements;

h. process complexity governing operating personnel requirements;

i. environmental impact on present and future adjacent land use;

j. sewage characteristics and the results of any treatability or pilot plant studies; and

k. reliability of the process and the potential for malfunctions or bypassing needs.

l. the possible effect of rising sea level on the treatment plant and its outfall.

4.5.2 Engineering Data for New Process Evaluation

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

The reviewing authority may require the following:

a. monitoring observations, including tests results and engineering evaluations, demonstrating the efficiency of such processes;

b. detailed description of the test methods;

c. testing, including appropriately 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; and

d. other appropriate information.

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

4.5.3 Design Loads

4.5.3.1 Design Period

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

- population growth rates;
- prevailing financing interest rates;
- inflation rates;
- ease of expansion of facilities; and
- time requirements for design and construction or expansion.

Wherever possible, sewage treatment plants should be designed for the flows expected to be received 20 years hence, under normal growth conditions. In certain cases, where it can be shown that staging of construction will be economically advantageous, lesser design periods may be used provided it can be demonstrated that the required capacity can be "on line" when needed.

4.5.3.2 Hydraulic Design

Flow conditions critical to the design of the treatment plant are described in Section 1.3.5.8.

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

The treatment plant design flow selected shall meet the appropriate effluent and water quality standards that are set forth in the permit to operate. The design of treatment units that are not subject to peak hourly flow requirements shall be based on the design average flow. For plants subject to high wet weather flows or overflow detention pumpback flows, the design maximum day flows that the plant is to treat on a sustained basis should be specified.

4.5.3.2.1 New Systems

Hydraulic Capacity for Wastewater Facilities to serve New Collection Systems.

i) The sizing of wastewater facilities receiving flows from new wastewater collection systems shall be based on an average daily flow of 340 ℓ per capita plus wastewater flow from industrial plants and major institutional and commercial facilities unless water use data or other justification upon which to better estimate flow is provided.

ii) The 340 l/cap·d figure shall be used, in conjunction with an extraneous flow allowances (see Section 2.3) intended to cover infiltration.

iii) If the new collection system is to serve existing development the likelihood of I/I contributions from existing service lines and non-wastewater connections to those service lines shall be evaluated and wastewater facilities designed accordingly.

4.5.3.2.2 Existing Systems

Hydraulic Capacity for Wastewater Facilities to serve existing Collection Systems:

i) Projections shall be made from actual flow data to the extent possible.

ii) The probable degree of accuracy of data and projections shall be evaluated. This reliability estimation should include an evaluation of the accuracy of existing data, as well as an evaluation of the reliability of estimates of flow reduction anticipated due to infiltration/inflow (I/I) reduction or flow increases due to elimination of sewer bypasses and backups.

iii) Critical data and methodology used shall be included. It is recommended that graphical displays of critical peak wet weather flow data (refer to Section 1.3.5.8) be included for a sustained wet weather flow period of significance to the project.

iv) At least one year's flow data should be taken as the basis for determining the various critical flow conditions.

4.5.3.2.3 Wet Weather Flows

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

4.5.3.2.4 Flow Equalization

Facilities for the equalization of flows and organic shock loads shall be considered at all plants which are critically affected by surge loadings. The sizing of the flow equalization facilities should be based on data obtained herein and from Section 2.3.

4.5.3.3 Organic Design Loads

4.5.3.3.1 Organic Load Definitions and Identification

The following organic loads for the design year shall be identified and used as a basis for design of wastewater treatment facilities. Where any of the terms defined in this Section are used in these design standards, the definition contained in this Section applies.

a. Biochemical Oxygen Demand Defined

The 5-day Biochemical Oxygen Demand (BOD_5) is defined as the amount of oxygen required to stabilize biodegradable organic matter under aerobic conditions within a five day period in accordance with latest edition of **Standard Methods**. Total 5-day Biochemical Oxygen demand ($TBOD_5$) is equivalent to BOD_5 and is sometimes used in order to differentiate carbonaceous plus nitrogenous oxygen demand from strictly carbonaceous oxygen demand.

The carbonaceous 5-day Biochemical Oxygen Demand ($CBOD_5$) is defined as BOD_5 less the nitrogenous oxygen demand of the wastewater. See the latest edition **Standard Methods**.

b. Design Average BOD_5

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

c. Design Maximum Day BOD_5

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

d. Design Peak Hourly BOD_5

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

4.5.3.3.2 New Systems

Organic Capacity of Wastewater Treatment Facilities to Serve New Collection Systems.

a. Domestic waste treatment design shall be on the basis of at least 0.08 kg of BOD_5 per capita per day and 0.09 kg of suspended solids per capita per day, unless information is submitted to justify alternate designs.

b. When garbage grinders are used in areas tributary to a domestic treatment plant, the design basis should be increased to 0.10 kg of BOD_5 per capita per day and 0.11 kg pounds of suspended solids per capita per day.

c. Where appreciable amounts of industrial waste are included, consideration shall be given to the character of wastes in the design of the plant.

d. Data from similar municipalities may be utilized in the case of new systems. However, thorough investigation that is adequately documented shall be provided to the reviewing authority to establish the reliability and applicability of such data.

4.5.3.3.3 Existing Systems

Design of Organic Capacity of Wastewater Treatment Facilities to Serve Existing Collection Systems:

a. Projections shall be made from actual wasteload data to the extent possible. Laboratory analyses shall be made on composite samples taken over 24 hour periods. This data shall include composite samples for the maximum significant period of sewage and industrial waste discharge, and shall cover a significant period of time to be representative of actual conditions. It is recommended that the designing engineer confer with the reviewing agency for details concerning the collection and analysis of samples. Recognition should be given to flow patterns for institutions, schools, motels, etc.

b. Projections shall be compared to Section 4.5.3.3.2 and an accounting made for significant variations from those values.

c. Impact of industrial sources shall be documented. For projects with significant industrial contributions, pretreatment may be required.

4.5.3.4 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.

4.5.3.5 Design by Analogy

Data from similar municipalities may be utilized in the case of new systems; however, a thorough investigation that is adequately documented shall be provided to the reviewing authority to establish the reliability and applicability of such data.

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

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

Sewage Pumping Stations

- peak hourly flow rate.

Screening

- peak flow rate.

Grit Removal

- peak flow rate, peak grit loading rate.

Primary Sedimentation

- peak flow rate, peak suspended solids loading rate.

Aeration (without nitrification)

- average diurnal BOD_5 loading rate is usually sufficient with predominantly domestic wastes, but the presence of significant industrial waste loadings may create sufficient diurnal variations to warrant consideration. Daily or seasonal variations in domestic and/or industrial BOD loading rates should be taken into consideration. Except for short detention treatment systems, such as contact stabilization or high rate processes, hydraulic detention time is seldom critical.

Aeration (with nitrification)

- average diurnal BOD_5 loading rate is usually sufficient with predominantly domestic wastes, but the presence of significant industrial waste loadings may create sufficient BOD diurnal variations to warrant consideration. Diurnal peak flow rate and diurnal peak ammonia (total Kjeldahl nitrogen with extended aeration) loading rates must be designed for. Daily or seasonal variations in BOD_5, ammonia, total Kjeldahl (with extended aeration) and peak flow rates should also be taken into consideration.

Secondary Sedimentation

- peak hourly flow rate or peak solids loading rate, whichever governs.

Sludge Return

- capacity requirements will vary with the treatment system (see Section 6.1.4).

Disinfection Systems

- peak flow rate.

Chemical Feed Systems

- peak flow rate.

Effluent Filtration

- peak flow rate, peak solids loading rate.

Outfall Sewer

- peak flow rate.

Sludge Treatment (digestion, thickening, dewatering, incineration, etc.)

- average loading rates (hydraulic, total solids, volatile solids) unless sustained peaks are of significance to the individual treatment process.

Effluent Retention Pond

- average daily flow rate for the anticipated low flow period (in a low flow receiving stream)

4.5.4 Conduits

All piping and channels should be designed to carry the maximum expected flows. The incoming sewer should be designed for unrestricted flow. Bottom corners of the channels must be filleted. Conduits shall be designed to avoid creation of pockets and corners where solids can accumulate. Suitable gates should be placed in channels to seal off unused sections which might accumulate solids. The use of shear gates or stop planks is permitted where they can be used in place of gate valves or sluice gates. Non-corrodible materials shall be used for these control gates.

4.5.5 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. Appropriate flow measurement shall be incorporated in the flow division control design.

4.5.6 Wastewater Flow Measurement

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

4.5.6.1 Location

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

a. Plant influent or effluent flow;

   If influent flow is significantly different from effluent flow, both shall be measured. This would apply for installations such as stabilization ponds, and plants with excess flow storage or flow equalization;

b. Bypass flow around sewage treatment plant;

c. Other flows required to be monitored under the provisions of the permit to operate; and

d. Other flows such as returned activated sludge, waste activated sludge, recirculation, and recycle required for plant operational control.

4.5.6.2 Facilities

Indicating, totalizing, and recording flow measurement devices shall be provided for all mechanical plants. Flow measurement facilities for stabilization pond systems shall not be less than elapsed time meters used in conjunction with pumping rate tests or shall be calibrated weirs. All flow measurement equipment must be sized to function effectively over the full range of flows expected and shall be protected against freezing.

4.5.6.3 Hydraulic Conditions

Flow measurement equipment including entrance 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 currants, air entrainment, etc., that upset the normal hydraulic conditions that are necessary for accurate flow measurement.

4.5.7 Component Back-up Requirements

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

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

4.5.8 Sampling Equipment

Effluent composite sampling equipment should be provided at all mechanical plants with a design average flow of 380 m^3/day or greater and at other facilities where necessary to meet "Approval/Permit to Operate" monitoring requirements. Composite sampling equipment shall also be provided as needed for influent sampling and for monitoring plant operations.

4.5.9 Plant Hydraulic Gradient

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

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

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

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

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

e. head losses due to obstructions in conduits;

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

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

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

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

j. head increases caused by pumping;

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

l. head allowances due to maximum water levels in receiving waters.

4.5.10 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.

4.6 PLANT DETAILS

4.6.1 Installation of Mechanical Equipment

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

4.6.2 By-Passes

4.6.2.1 General

Except where duplicate units are available, properly located and arranged by-pass structures shall be provided so that each unit of the plant can be removed from service independently. The by-pass 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. By-pass systems should also be constructed so that each unit process can be separately by-passed.

4.6.2.2 Unit By-Pass During Construction

Final plan documents shall include construction requirements as deemed necessary by the reviewing agency to avoid unacceptable temporary water quality degradation.

4.6.3 Overflows

If sewage entering the treatment plant must be pumped into the treatment units, an emergency overflow for the pumping station should be provided, if it is physically possible (reference to section 3.3.2). The purpose of this overflow is to prevent basement flooding by back-ups in the sewer system in the event of pumping station failure. Wherever possible, this overflow should be routed through the treatment plant disinfection systems and plant outfall sewer. If this is not possible, provision should be made for chlorination of such overflows. If chlorination is utilized, it may be necessary to dechlorinate to address concerns related to the negative environmental and health impacts of the release of chlorine into aquatic environments.

The overflow elevation and the method of activation should ensure that the maximum feasible storage of the wet well will be utilized before the controlled overflow takes place. The overflow facilities should at least be alarmed and equipped to indicate frequency and duration of overflows and provided with facilities to permit manual flow measurement. Automatic flow measurement and recording systems may be required in certain cases where effluent quality requirements dictate. Where the operator is signatory to a Shellfish Conditional Area Management Plan, notification and reporting requirements of the plan shall be met. All overflows should be recorded and reported to the regulatory agencies."

4.6.4 Drains

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

4.6.5 Construction Materials

Due consideration should be given to the selection of materials which are 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. Dissimilar metals should be avoided to minimize galvanic action.

4.6.6 Painting

The use of paints containing lead or mercury should be avoided. In order to facilitate identification of piping, particularly in the large plants, it is suggested that the different lines be colour-coded. The following colour scheme is recommended for purposes of standardization.

Raw sludge line - brown with black bands
Sludge recirculation suction line - brown with yellow bands
Sludge draw off line - brown with orange bands
Sludge recirculation discharge line - brown
Sludge gas line - orange (or red)
Natural gas line - orange (or red) with black bands
Nonpotable water line - blue with black bands
Potable water line - blue
Chlorine line - yellow
Sulphur Dioxide - yellow with red bands
Sewage (wastewater) line - grey
Compressed air line - green
Water lines for heating digesters or buildings - blue with a 150 mm red band spaced 760 mm apart

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

4.6.7 Operating Equipment

A complete outfit of tools and accessories and spare parts necessary for the plant operator's use shall be provided. A portable pump is desirable. Readily accessible storage space and work bench facilities shall be provided and consideration given to provision of a garage area which would also provide space for large equipment, maintenance and repair.

4.6.8 Grading and Landscaping

Upon completion of the plant, the ground should be graded. Concrete or gravel 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, sludge beds and intermittent sand filters, from surface wash. Provision should be made for landscaping, particularly when a plant is located near residential areas.

4.6.9 Erosion Control During Construction

Effective site erosion control shall be provided during construction as outlined in the Nova Scotia Department of the Environment publication, "Erosion and Sedimentation Control Handbook for Construction Sites". An approved erosion control plan is required before construction begins.

4.6.10 Cathodic Protection

Steel fabricated sewage treatment plants shall require cathodic protection for corrosion control as specified in Section 3.2.12.

4.7 PLANT OUTFALLS

4.7.1 Dilution

Outfall sewers shall consist of a completely piped system conforming to the requirements of Chapter 2 of these guidelines and shall not discharge into any ditch or watercourse in which adequate assimilative capacity is not available. In assessing the available assimilative capacity the proximity of other outfalls must be taken into consideration. The outfall sewer shall be designed to discharge to the receiving water in a manner acceptable to the reviewing authority. Consideration should be given to each of the following: a) utilization of cascade aeration of effluent discharge to increase dissolved oxygen levels and b) limited or complete across-stream dispersion as needed to protect aquatic life movement and growth in the immediate reaches of the receiving stream.

4.7.2 Outlet

The outfall sewer, where practicable, shall be extended to the low water level of the receiving body of water in such a manner to insure the satisfactory dispersion of the effluent thereto and insofar as practicable, it shall have its outlet submerged. Where greater depths are available, one metre should be the achieved depth of submergence.

4.7.3 Protection and Maintenance

The outfall sewer shall be so constructed and protected against the effects of flood water, tides, ice or other hazards as to reasonably insure its structural stability and freedom from stoppage.

A manhole should be provided at the shore end of all gravity outfall sewers extending into the receiving waters.

Hazards to navigation must be considered in designing outfall sewers.

4.7.4 Dispersion of Flow

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

4.7.5 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.

4.8 ESSENTIAL FACILITIES

4.8.1 Emergency Power Facilities

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:

- reliability of primary power source;
- 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;
- whether sewage enters the plant by gravity or is pumped;
- type of treatment provided;
- pieces of equipment which may become damaged or overloaded following prolonged power failure;
- assimilation capacity of the receiving waters and ability to withstand higher pollution loadings over short time periods; and
- other uses of the receiving water.

Each specific installation should provide for the following considerations:

- means for illuminating working areas to ensure safe working conditions; and

- 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.

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 have 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.

4.8.2 Water Supply

4.8.2.1 General

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

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

4.8.2.2 Direct Connections

Potable water from a municipal or separate supply may be used directly at points above grade for the following hot and cold supplies:

a. lavatory sink;
b. water closet;
c. laboratory sink;
d. shower;
e. drinking fountain;
f. eye wash fountain; and
g. 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 coils.

4.8.2.3 Indirect Connections

Where a potable water supply is to be used for any purpose in a plant other than those listed in Section 4.7.2.2, a break tank, pressure pump and pressure tank shall be provided. Water shall be discharged to the break tank through an air-gap at least 150 mm above the maximum flood line or the spill line of the tank, whichever is higher.

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

Consideration will also be given to backflow devices consisting of a system of check valves and relief valves which provide protection against backflow.

4.8.2.4 Separate Potable Water Supply

Where it is not possible to provide potable water from a public water supply, a separate well may be provided as long as sufficient pressure is available. Location and construction of the well should comply with requirements of the regulatory authorities. Requirements governing the use of the supply are those contained in Sections 4.8.2.2 and 4.8.2.3.

4.8.2.5 Separate Non-Potable Water Supply

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

4.8.3 Sanitary Facilities

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

4.8.4 Floor Slope

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

4.8.5 Stairways

Stairways should be installed with a slope of 30 to 35 degrees from the horizontal to facilitate carrying samples, tools, etc. All risers in a stairway should be of equal height. Minimum tread run shall not be less than 200 mm. The sum of the tread run and riser shall not be less than 430 mm nor more than 460 mm. A flight of stairs shall consist of not more than 3 m continuous rise without a platform. Stairways shall be installed wherever possible in lieu of ladders.

4.9 SAFETY

4.9.1 General

Adequate provision shall be made to effectively protect the operator and visitors from hazards. The following shall be provided to fulfil the particular needs of each plant:

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

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

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

d. First aid equipment;

e. "No Smoking" signs in hazardous areas;

f. Protective clothing and equipment, such as self-contained breathing apparatus, gas detection equipment, goggles, gloves, hard hats, safety harnesses, etc.;

g. Portable blower and sufficient hose;

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

i. Gas detectors;

j. 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.;

k. Adequate ventilation in pump station areas in accordance with Section 3.2.7.

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

m. Provisions for confined space entry in accordance with regulatory agency requirements.

4.9.2 Hazardous Chemical Handling

Reference should be made to Federal "Transportation of Dangerous Goods Act" and the Provincial "Dangerous Goods and Hazardous - Wastes Management Act".

4.9.2.1 Contaminant Materials

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

4.9.2.2 Primary Containment

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

4.9.2.3 Secondary Containment

Chemical storage areas shall be enclosed in dykes or curbs which are capable of containing 110% of the stored volume until it can be safely transferred to alternate storage or released to the wastewater at controlled rates which will not damage facilities, inhibit the treatment processes or contribute to stream pollution. Liquid polymer should be similarly contained to reduce areas with slippery floors, especially to protect travel-ways. Non-slip floor surfaces are desirable in polymer handling areas.

4.9.2.4 Underground Storage

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

4.9.2.5 Liquified Gas Chemicals

Properly designed isolated areas shall be provided for storage and handling of chlorine and sulphur dioxide and other hazardous gases. Gas detection unit, alarms, controls, safety devices, and emergency repair kits shall also be provided.

4.9.2.6 Eye Wash Fountains and Safety Showers

Eye wash fountains and safety showers utilizing potable water shall be provided in the laboratory and on each floor level or work location involving hazardous or corrosive chemical storage, mixing (or shaking), pumping, metering, or transportation unloading. These facilities are to be as close as practical to possible chemical exposure sites and are to be fully useful during all weather conditions, and shall be no more than 7.0 m from points of hazardous chemical exposure.

The eye wash fountains shall be supplied with water of moderate temperature (~~10~~ 16 to 25℃), suitable to provide 15 to 30 minutes of continuous irrigation of the eyes. Anti-scalding devices should be provided as required to maintain a relatively constant temperature. Portable, self-contained eyewash stations may be used to supplement plumbed eyewash stations, but in no way replace them. The main purpose of such a portable unit is to provide immediate flushing. Once accomplished, the user should proceed to a plumbed eyewash station and flush for the required flushing/rinsing period.

The emergency showers shall be capable of discharging 1.3 litres per second of water at moderate temperature (15 to 25℃) at pressures of 140 to 350 kPa, suitable to provide a minimum of 15 minutes of continuous flow or as required by the MSDS for the types of hazardous chemical being handled at the facility.

4.9.2.7 Splash Guards

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

4.9.2.8 Piping, Labelling, Coupling Guards, Location

All piping containing or transporting corrosive or hazardous chemicals shall be identified with labels every 3 m and with at least two labels in each room, closet or pipe chase. Colour coding may also be used but is not an adequate substitute for labelling. All connections (flanged or other type), except adjacent to storage or feeder areas, shall have guards which will direct any leakage away from space occupied by personnel. Pipes containing hazardous or corrosive chemicals should not be located above shoulder level except where continuous drip collection trays and coupling guards will eliminate chemical spray or dripping on to personnel.

4.9.2.9 Protective Clothing or Equipment

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

a. self-contained air supply system recommended for protection against chlorine;

b. chemical worker's goggles or other suitable goggles (safety glasses are insufficient);

c. face masks or shields for use over goggles;

d. rubber gloves;

e. rubber aprons with leg straps;

f. rubber boots (leather and wool clothing should be avoided near caustics); and

g. safety harness and line.

h. dust mask to protect the lungs in dry chemical areas.

4.9.2.10 Warning Systems and Signs

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

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

4.9.2.11 Dust Collection

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

4.9.2.12 Container Identification

The identification and hazard warning data included on shipping containers, when received, should appear on all containers (regardless of size or type) used to store, carry or use a hazardous substance. Sewage and sludge sample containers should be adequately labelled. Below is a suitable label for sewage sample:

RAW SEWAGE **Sample point no.** **Contains Harmful Bacteria.** **May contain hazardous or toxic material.**

Do not drink or swallow.

Avoid contact with openings or breaks in the skin.

4.10 LABORATORY

4.10.1 General

All treatment works shall include a laboratory for making the necessary analytical determinations and operating control tests, except in individual situations where the omission of a laboratory is approved by the reviewing agency. The laboratory shall have sufficient size, bench space, equipment and supplies to perform all self-monitoring analytical work required by the Permit to Operate and to perform the process control tests necessary for good management of each treatment process included in the design.

The facilities and supplies necessary to perform analytical work to support industrial waste control programs will normally be included in the same laboratory. The laboratory size and arrangement must be sufficiently flexible and adaptable to accomplish these assignments. The layout should consider future needs for expansion in the event that more analytical work is needed. Laboratory instrumentation and size should reflect treatment plant size, staffing requirements, and process complexity. Experience and training of plant operators should also be assessed in determining treatment plant laboratory needs.

Treatment plant laboratory needs may be divided into the following three general categories:

I. Plants performing only basic operational testing; this typically includes pH, temperature, and dissolved oxygen;

II. Plants performing more complex operational and permit laboratory tests including biochemical oxygen demand, suspended solids, and fecal coliform analysis, and;

III. Plants performing more complex operational, permit, industrial pretreatment, and multiple plant laboratory testing.

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

4.10.2 Category I: Plants performing only basic operational testing.

4.10.2.1 Location and Space

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

4.10.2.2 Design and Materials

The facility shall provide for electricity, water, heat, sufficient storage space, a sink, and a bench top. The lab components need not be of industrial grade materials. Laboratory equipment and glassware shall be of types recommended by Standard Methods for the Examination of Water and Wastewater and the reviewing authority.

4.10.3 Category II: Plants performing more complex operational and permit laboratory tests including biochemical oxygen demand, suspended solids, and fecal coliform analysis.

4.10.3.1 Location and Space

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

4.10.3.2 Floors

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

4.10.3.3 Cabinets and Bench Tops

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

4.10.3.4 Fume Hoods, Sinks, and Ventilation

4.10.3.4.1 Fume Hoods

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

4.10.3.4.2 Sinks

A laboratory grade sink and drain trap shall be provided.

4.10.3.4.3 Ventilation

Laboratories should be air conditioned. In addition, separate exhaust ventilation should be provided.

4.10.3.5 Balance and Table

An analytical balance of the automated digital readout, single pan 0.1 mg sensitivity type shall be provided. A heavy special-design balance table which will minimize vibration of the balance is recommended. It shall be located as far as possible from windows, doors, or other sources of drafts or air movements, so as to minimize undesirable impacts from these sources upon the balance.

4.10.3.6 Equipment, Supplies, and Reagents

The laboratory shall be provided with all of the equipment, supplies, and reagents that are needed to carry out all of the facility's analytical testing requirements. If any required analytical testing produces malodorous or noxious fumes, the engineer should verify that the in-house analysis is more cost-effective than use of an independent off-site laboratory. Composite samples may be required to satisfy permit sampling requirements. Permit to operate, process control, and industrial waste monitoring requirements should be considered when specifying equipment needs. References such as Standard Methods for the Examination of Water and Wastewater and the U.S.E.P.A. Analytical Procedures Manual should be consulted prior to specifying equipment items.

4.10.3.7 Utilities

4.10.3.7.1 Power Supply

Consideration should be given to providing line voltage regulation for power supplied to laboratories using delicate instruments.

4.10.3.7.2 Laboratory Water

Reagent water of a purity suitable for analytical requirements shall be supplied to the laboratory. In general, reagent water prepared using an all glass distillation system is adequate. However, some analyses require deionization of the distilled water. Consideration should be given to softening the feed water to the still.

4.10.3.8 Safety

4.10.3.8.1 Equipment

Laboratories shall provide as a minimum the following: first aid equipment; protective clothing including goggles, gloves, lab aprons, etc.; and a fire extinguisher.

4.10.3.8.2 Eyewash Fountains and Safety Showers

Eyewash fountains and safety showers shall be provided as per Section 4.9.2.6.

4.10.4 Category III: Plants performing more complex operational, permit, industrial pretreatment and multiple plant laboratory testing.

4.10.4.1 Location and Space

The laboratory should be located at the treatment site on ground level, with environmental control as an important consideration. It shall be located away from vibrating, noisy, high temperature machinery or equipment which might have adverse effects on the performance of laboratory staff or instruments.

The laboratory facility needs for Category III plants should be described in the engineering design report or facilities plan. The laboratory floor space and facility layout should be based on an evaluation of complexity, volume, and variety of sample analyses expected during the design life of the plant including testing for process control, industrial pretreatment control, user charge monitoring, and the permit to operate monitoring requirements.

Consideration should be given to the necessity to provide separate (and possibly isolated) areas for some special laboratory equipment, glassware, and chemical storage. At large plants, office and administrative space needs should be considered.

For less complicated laboratory needs bench-top working surface should occupy at least 35 percent of the total laboratory floor space. Additional floor and bench space should be provided to facilitate performance of analysis of industrial wastes, as required by the permit to operate and the utility's industrial waste pretreatment program. Ceiling height should be adequate to provide for the installation of wall mounted water stills, deionizers, distillation racks, hoods, and other equipment with extended height requirements.

4.10.4.2 Floor and Doors

4.10.4.2.1 Floors

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

4.10.4.2.2 Doors

Two exit doors should be located to permit a straight egress from the laboratory, preferable at lease one to outside the building. Panic hardware should be used. They should have large glass windows for easy visibility of approaching or departing personnel.

Automatic door closers should be installed; swinging doors should not be used.

Flush hardware should be provided on doors if cart traffic is anticipated. Kick plates are also recommended.

4.10.4.3 Cabinets and Bench Tops

4.10.4.3.1 Cabinets

Wall-hung cabinets are useful for dust-free storage of instruments and glassware. Units with sliding glass doors are preferable. A reasonable proportion of cupboard style base cabinets and drawer units should be provided.

Drawers should slide out so that entire contents are easily visible. They should be provided with rubber bumpers and with stops which prevent accidental removal. Drawers should be supported on ball bearings or nylon rollers which pull easily in adjustable steel channels. All metals drawer fronts should be double-wall construction.

All cabinet shelving should be acid resistant and adjustable. The laboratory furniture shall be supplied with adequate water, gas, air, and vacuum service fixtures, traps, strainers, plugs, and tailpieces, and all electrical service fixtures.

4.10.4.3.2 Bench Tops

Bench tops should be constructed of materials resistant to attacks from normally used laboratory reagents. Generally, bench-top height should be 900 mm. However, areas to be used exclusively for sit-down type operations should be 760 mm high and include kneehole space. Twenty-five millimetre overhangs and drip grooves should be provided to keep liquid spills from running along the face of the cabinet. Tops should be furnished in large sections, 32 mm thick. They should be field-jointed into a continuous surface with acid, alkali, and solvent-resistant cements which are at least as strong as the material of which the top is made.

4.10.4.4 Hoods

4.10.4.4.1 General

Fume hoods to promote safety and canopy hoods over heat-releasing equipment shall be provided.

4.10.4.4.2 Fume Hoods

a. Location

Fume hoods should be located where air disturbance at the face of the hood is minimal. Air disturbance may be created by persons walking past the hood; by heating, ventilating, or air-conditioning systems; by drafts from opening or closing a door, etc.

Safety factors should be considered in locating a hood. If a hood is situated near a doorway, a secondary means or egress must be provided. Bench surfaces should be available next to the hood so that chemicals need not be carried long distances.

b. Design and Material

The selection, design, and materials of construction of fume hoods and their appropriate safety alarms must be made by considering the variety of analytical work to be performed. The characteristics of the fumes, chemicals, gases, or vapours that will or may be released by the activities therein should be considered. Special design and construction is necessary if perchloric acid use is anticipated. Consideration should be given to providing more than one fume hood to minimize potential hazardous conditions throughout the laboratory.

Fume hoods are not appropriate for operation of heat-releasing equipment that does not contribute to hazards, unless they are provided in addition to those needed to perform hazardous tasks.

c. Fixtures

One sink should be provided inside each fume hood. A cup sink is usually adequate.

All switches, electrical outlets, and utility and baffle adjustment handles should be located outside the hood. Light fixtures should be explosion-proof.

d. Exhaust

Twenty-four hour continuous exhaust capability should be provided. Exhaust fans should be explosion-proof. Exhaust velocities should be checked when fume hoods are installed.

4.10.4.4.3 Canopy Hoods

Canopy hoods should be installed over the bench-top areas where hot plate, steam bath, or other heating equipment or heat-releasing instruments are used. The canopy should be constructed of heat and corrosion resistant material.

4.10.4.5 Sinks, Ventilation, and Lighting

4.10.4.5.1 Sinks

The laboratory should have a minimum of two sinks (not including cup sinks). At least one of them should be a double-well sink with drainboards. Additional sinks should be provided in separate work areas as needed, and identified for the use intended.

Sinks should be made of epoxy resin or plastic materials highly resistant to acids, alkalies, solvents, and salts, and should be abrasion and heat resistant, non-absorbent, and light in weight. Traps should be made of glass, plastic, or lead and easily accessible for cleaning. Waste openings should be located toward the back so that a standing overflow will not interfere.

All water fixtures on which hoses may be used should be provided with reduced zone pressure backflow preventers to prevent contamination of water lines.

4.10.4.5.2 Ventilation

Laboratories should be separately air conditioned, with external air supplied for one hundred percent make-up volume. In addition, separate exhaust ventilation should be provided. Ventilation outlet locations should be remote from ventilation inlets. Consideration should be given to providing dehumidifiers.

4.10.4.5.3 Lighting

Good lighting, free from shadows, must be provided for reading dials, meniscuses, etc., throughout the laboratory.

4.10.4.6 Balance and Table

An analytical balance of the automatic, digital readout, single pan, 0.1 mg sensitivity type shall be provided. A heavy special-design balance table which will minimize vibration of the balance is needed. It shall be located as far as practical from windows, doors, or other sources of drafts or air movements, so as to minimize undesirable impacts from these sources upon the balance.

4.10.4.7 Equipment, Supplies, and Reagents

The laboratory shall be provided with all of the equipment, supplies, and reagents that are needed to carry out all of the facility's analytical testing requirements. Composite samplers may be required to satisfy permit sampling requirements. Permit to operate, process control, and industrial waste monitoring requirements should be considered when specifying equipment needs. Reference such as Standard Methods for the Examination of Water and Wastewater and the U.S.E.P.A. Analytical Procedures Manual should be consulted prior to specifying equipment items.

4.10.4.8 Utilities and Services

4.10.4.8.1 Power Supply

Consideration should be given to providing line voltage regulation for power supplied to laboratories using delicate instruments.

4.10.4.8.2 Laboratory Water

Reagent water of a purity suitable for analytical requirements shall be supplied to the laboratory. In general, reagent water prepared using an all glass distillation system is adequate. However, some analyses require deionization of the distilled water. Consideration should be given to softening water to the still.

4.10.4.8.3 Gas and Vacuum

Natural or LP gas should be supplied to the laboratory. Digester gas should not be used.

An adequately-sized line source of vacuum should be provided with outlets available throughout the laboratory.

4.10.4.9 Safety

4.10.4.9.1 Equipment

Laboratories shall be provide the following: first aid equipment; protective clothing and equipment such as goggles, safety glasses, full face shields, gloves, etc.; fire extinguishers; chemical spill kits; posting of "No Smoking" signs in hazardous area; and appropriately placed warning signs for slippery areas, non-potable water fixtures, hazardous chemical storage areas, flammable fuel storage areas, etc.

4.10.4.9.2 Eyewash Fountains and Safety Showers

Eyewash fountains and safety shall be provided as per Section 4.9.2.6.

4.11 SEWAGE WORKS – INSTRUMENTATION AND CONTROLS

Several factors should be considered when developing a plan for the instrumentation and controls for a wastewater treatment facility. Monitoring requirements vary depending on the type of facility being considered and its location; this will impact on the selection and type of instrumentation being considered. Instrumentation and control requirements will also depend on the size of the plant, and as each treatment process has its own set of conditions to be monitored and controlled there will be different technical requirements to be met. In general, instrumentation and control should provide efficient and safe automatic and manual operation of all plant systems with a minimum of operator effort. Automatic systems should also be provided with manual back-up systems.

The design shall have provision for local control systems where parts of the plant may be operated or controlled from a remote location. The local control stations should include provision for preventing the operation of equipment from remote locations.

When making decisions relating to instrumentation and control, the following factors should be considered:

1. Plant size and complexity;
2. Regulatory requirements;
3. Hours of attended operation;
4. Potential chemical and energy savings;
5. Primary element reliability
6. Primary element location
7. Whether controls should be manual or automatic; and
8. The data storage and recording requirements and whether data acquisition should be central or distributed
9. Safety

For effective operation of larger wastewater treatment facilities the following parameters should be measured, some may not be required for smaller facilities.

1. Flowrate for raw sewage, by-pass flows, final effluent flow;
2. Return Activated Sludge (RAS) flows, Waste Activated Sludge (WAS) flows;
3. Raw and digested sludge flow, digester supernatant flows;
4. Chemical dosage, digester gas production;
5. Hazardous gas monitoring;
6. Anaerobic digester temperature;
7. Dissolved oxygen levels; and
8. Sludge blanket levels and sludge concentrations.

4.11.1 Types of Instruments

The different types of instruments that may be required to measure the previously mentioned parameters are classified as primary element devices, which alter a signal from a physical process to make it suitable for use by a transmitter. These devices are broken down into function groups with a brief description of the process application.

4.11.1.1 Flow Measurement

4.11.1.1.1 Magnetic Flowmeters (Mag Meters)

Liner and Electrode Materials - The liner for the meter can vary depending on the application being considered. In applications where moderate amounts of abrasion are likely to occur, one of the following materials may be selected; Polyurethane, Butyl rubber, Neoprene or Polytetrafluoroethylene. In applications where corrosion is likely to occur, one of the following materials may be selected; ceramic or Polytetrafluoroethylene. Stainless steel electrode material should be used for applications where corrosion is not likely to present a problem. Hastelloy electrode material should be used for applications where corrosion is likely to present a problem.

Installation - Installation of magnetic flow meters generally require five straight pipe diameters upstream of the meter and three down stream of the meter free of valves or fittings. Meters may be installed on horizontal, vertical or sloping lines. It is essential to keep the electrodes in the horizontal plane to assure uninterrupted contact with the fluid or slurry being metered. The operating velocity required for these meters will fall into the range or (1 to 10 m/s) for non-solids bearing liquids and (1.5 to 7.5 m/s) for solids bearing liquids. When used to meter liquids containing solids, a continuous electrode cleaner or clean out tee should be installed.

Applications - These meters are suitable for Influent Wastewater, Primary Sludge, RAS, WAS, Digested Sludge and Final Effluent. These meters should not be used for Digester Gas or liquid streams with a solids content greater than 10% by weight.

4.11.1.1.2 *Ultrasonic Flowmeters*

Flowmeter Construction - The flowmeter usually consists of an electronics housing, transducers and pipe section. These can in many cases be fitted to existing pipes either by drilling holes for the transducer hardware or by application of external transducers to the outside of the pipe. When installed on existing pipes, the existing pipe material should be checked to assure it will not dampen the sonic signal as this will adversely affect performance.

Installation - The installation of Ultrasonic flow meters generally require ten to twenty straight pipe diameters upstream of the meter and five down stream of the meter free of valves or fittings. Meters can be installed on horizontal, vertical or sloping lines as long as the pipe sections are always full. The operating velocity required for these meters will fall into the range of (1 to 10 m/s).

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

4.11.1.1.3 *Turbine Flowmeters*

Flowmeter Construction - The flowmeter usually consists of meter body with rotor blades and a magnetic pickup. The pickup is often connected to electronic display units or a totalizer.

Installation - Installation of turbine flow meters generally require a minimum of stream of the meter free of valves or fittings. Meters may be installed on horizontal or vertical pipelines.

Applications - Turbine flow meters are recommended for applications involving natural gas, compressed digester gas.

4.11.1.1.4 *Flumes and Weirs (Parshall Flume)*

Installation - The flume will be affected by upstream channel arrangement and it is recommended that there be at least ten channel widths upstream. The flume must also be installed carefully to make certain that it is level.

Applications - Flumes and weirs are customarily used to measure flows in open channels. They are recommended for applications involving open channel flow measurement.

4.11.1.2 *Suspended Solids Measurement (Turbidity)*

Installation - Installation details for turbidity analyzers are unique to each manufacturer. The manufacturer's recommendations should be followed.

Applications - Turbidity analyzers are recommended for applications involving suspended solids concentrations less than 100 mg/l.

4.11.1.3 Suspended Solids Measurement (Optical)

Installation - Installation details for optical analyzers are unique to each manufacturer. The manufacturer's recommendations should be followed.

Applications - Optical analyzers are recommended for applications involving solids concentrations from 20 mg/l to 8%. Examples are, RAS, WAS and mixed liquor.

4.11.1.4 Dissolved Oxygen Measurement (Galvanic)

Installation - Installation details for dissolved oxygen analyzers are usually related to the choice of placement of the analyzer in the process fluid. The analyzers generally require fairly frequent maintenance and this should be considered in determining the location for installation.

Applications - Oxygen analyzers are recommended for applications involving oxygen concentrations from 0 to 20 mg/l.

4.11.1.5 Level Measurement

4.11.1.5.1 Ultrasonic Sensor

Installation - The mounting location of the sensor is determined from restrictions established by the manufacturer. Typically the sensor must be mounted a minimum distance above the high liquid level and should be located away from tank walls or other obstructions that may cause false echoes.

Applications - This type of level element may be used in many level and flow applications; it is not recommended in locations where foam is dense and persistent.

4.11.1.5.2 Float

Installation - Float switches are normally located in a stilling well when turbulence is expected.

Applications - Float switches are commonly used for high and low level alarms and for controlling pump starts and stops.

4.11.1.5.3 Capacitance

Installation- The installation practices can vary and the manufactures recommended installation should be used.

Applications- May be used in applications that require continuous level measurement and also as switches for alarms or start/stop control.

4.11.1.6 Pressure Measurement

4.11.1.6.1 Bourdon Tubes

Installation - The installation practice should include the use of block and bleed valves.

Applications - May be used in applications that require pressure indication. Pressure range 0 to 35000 kPa.

4.11.1.6.2 Bellows

Installation - The installation practice should include the use of block and bleed valves.

Applications - May be used in applications that require pressure indication. Pressure range 0 to 2000 kPa.

4.11.1.6.3 Diaphragms

Installation - The installation practice should include the use of block and bleed valves. Transmitters should be installed according to manufacturer's recommendations. Temperature extremes should be avoided and location should be as close as possible to the process measure site.

Applications - May be used in applications that require pressure indication or transmitter output. Pressure range 0 to 3500 kPa.

4.11.1.7 Temperature Measurement

4.11.1.7.1 Thermocouples

Installation - The thermocouple should be selected with care to assure that the appropriate device is chosen for the given temperature range. Installation with a thermowell is advised.

Applications - Thermocouples are suitable for most temperature measurement applications.

4.11.1.7.2 Resistance Temperature Detector

Installation - The resistance detector should be selected with care to assure that the appropriate device is chosen for the given temperature range. Installation with a thermowell is advised.

Applications - Resistance detectors are suitable for temperature measurement applications with ranges of 0 to 300°C.

4.11.1.7.3 Thermistor

Installation - The Thermistor should be selected to assure that the device is appropriate for the temperature range. Installation with a thermowell is advised.

Applications - Thermistors are suitable for temperature measurement applications with ranges of 0 to 300°C.

4.11.1.7.4 Thermal Bulb

Installation - No special installation requirements.

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

4.11.2 Process Controls

4.11.2.1 Lift Stations

Lift stations require simple and dependable instrumentation and control systems. The parameters that should be monitored are level, flow, pressure, temperatures, hazardous gas levels, as well as status and alarm conditions. The monitoring and control requirements will vary for each individual case based on the size, location, and economic considerations.

4.11.2.2 Level Control

Lift stations vary in size and storage capacity but generally they require similar controls. The level in the wet well increases to the point where a duty pump will be required to start, a lag and follow pump may be started if the level continues to increase. Pumping continues until a pump stop level is reached at which time the duty pump stops, or a series of stop levels will be reached and the lag and follow pumps stop prior to the duty pump. The pump start/ stop control can be performed using any one of several level elements.

When variable speed pumps are used there are several ways in which the pump can be controlled. These generally are controlled to maintain a level set point in the wetwell. This requires a feedback type of control in which the measured variable (level) is compared to a set point value and the final control element is modulated in order to maintain the set point value. Level control of this nature require reliable analog level measurement if it is to function properly. Regardless of the type of level control selected, the system should include a separate low level lockout and high level alarm.

4.11.2.2.1 Flow Monitoring

The flow metering element should be selected carefully to ensure that there are no obstructions where clogging may occur. Provision should be made so that the flow-metering element can be bypassed or isolated for routine maintenance activities. The flow-metering device should be connected to either the control system or to a recording and totalizing device or both. This provides for a record of flows out of the lift station. It can also be used to help identify possible problems in the discharge piping or force main.

4.11.2.2.2 Pressure Monitoring

Monitoring of the system discharge pressure can be useful in identifying possible problems in the discharge piping or force main and in monitoring pump performance. The pressure-metering device should be connected to either the control system or to a recording device or both.

4.11.2.2.3 Pumps and Motors

The following parameters should be monitored:

1. Pump bearing temperature;
2. Pump bearing vibration;
3. Pump speed for variable speed applications;
4. Pump discharge pressure;
5. Motor voltage and current;
6. Motor hours of operation;
7. Motor bearing temperature; and
8. Motor windings temperature.

4.11.2.2.4 Alarms

Lift stations should be alarmed as outlined in Section 3.2.12.

4.11.2.3 Mechanical Bar Screens

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

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

2. Automatic activation by differential level. This method uses the differential level across the screen to provide the start condition. The screen should run at least one complete screen cycle before stopping. The screen can be called to stop when the differential level is returned to a nil value, the final stop should be controlled using a sensor to determine cycle completion (i.e. limit switch, proximity sensor, timer). In addition, a timer should be provided to initiate a cleaning cycle at regular intervals regardless of actual head loss. When this method is employed, there should be an alarm signal with a head loss set at a point higher than the automatic start of the mechanical bar screen.

3. Automatic activation by timer with differential level as emergency start condition. This method uses the differential level across the screen to provide secondary start condition. The screen should run at least one complete screen cycle before stopping. The stop signal should be controlled using a sensor to determine cycle completion (i.e. limit switch, proximity sensor, timer). When this method is employed there should be an alarm signal with a head loss set at a point higher than the automatic start of the mechanical bar screen.

4.11.2.4 Primary Treatment

4.11.2.4.1 Raw Sludge Pumping

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

1. Automatic or manual selection of duty pump;
2. On line sludge density metering for control and monitoring;
3. On line sludge flow monitoring and totalization;
4. On line adjustable sludge density control;
5. Individually selectable hopper pumping controls where required;
6. Manual override for automatic controls;
7. On line sludge blanket level monitoring and alarming;
8. On line sludge pump monitoring and control;
9. Sludge density feedback control for variable speed pumping with manual override;
10. On line sludge pump speed monitoring and control with manual override; and
11. On line monitoring and control of primary tank scraper mechanisms.

4.11.2.4.2 Scum Pumping

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

1. Automatic or manual selection of duty pump;
2. Manual override for automatic controls;
3. On line sludge blanket level monitoring and alarming;
4. Automatic controls consisting of high and low scum tank level for starting and stopping scum pumps;
5. High scum tank level alarm;
6. On line scum pump speed monitoring and control with manual override; and
7. Scum tank flushing system for scum tank cleaning

4.11.2.5 Secondary Treatment

4.11.2.5.1 Dissolved Oxygen (DO) Control

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

1. Closed Loop Control (Feedback Control) - Closed loop control consists of on line dissolved oxygen analyzers providing feedback control to an airflow control device. The dissolved oxygen reading is compared to the dissolved oxygen set point. The resultant error signal is used to increase or decrease the rate of air flow to the aeration tanks. Automatic dissolved oxygen control should always be equipped with manual override.

2. Feed Forward Control - Feed forward control consists of a fixed volume of air being delivered to the aeration tanks for a given flowrate. This system may utilize on line dissolved oxygen analyzers but these are used for monitoring only and do not provide feedback to the air flow control elements. Process status and alarms should be provided for dissolved oxygen level, blower operating parameters, air flow control elements.

4.11.2.5.2 Return Activated Sludge Control

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

1. Automatic or manual selection of duty pump;
2. Variable speed pumping;
3. Return activated sludge flow monitoring;
4. Feedback control to match pumping rates to flow set points;
5. Individual control of sludge return rate from individual final clarifiers;
6. Manual override for automatic controls; and
7. On line monitoring of return sludge flowrate, pump speed and status.

4.11.2.5.3 Waste Activated Sludge Control

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

1. Automatic or manual selection of duty pumps;
2. Variable speed pumping;
3. Waste Activated Sludge flow monitoring;
4. Feedback control to match pumping rates to flow set points;
5. Manual override for automatic controls; and
6. On line monitoring of Waste Sludge flowrate, pump speed and status.

4.11.2.5.4 Chemical Control System

Chemical addition consists of a feeder or chemical metering pump that will dose at a fixed ratio to the influent or effluent flow of the plant, with no analyzer or feedback control. More specific chemical dosing may also be based on such things as return sludge flowrate. Chemical dosing requirements will vary widely depending on performance requirements and the specific process being utilized.

4.11.2.5.5 Disinfection Control Systems (Ultra Violet)

The disinfection of final plant effluent utilizing ultra violet light consists of a feed forward control system. This consists of a series of lamps and or lamp channels that are turned on based on effluent flow of the plant, UV transmittance analyzers may be utilized for monitoring system performance but are not generally employed in feedback control.

4.11.2.6 Control and Monitoring Systems

Control and monitoring systems can be a conventional system with recorders, indicators, switches, push buttons, indicating lights, control panels, etc. or it can be a computerized control system that utilizes various configurations of hardware and software to provide the control required. Computerized systems can be separated into two groups, PLC (Programmable Logic Controller) Systems and Distributed Control Systems.

4.11.2.6.1 Conventional Relay Control Systems

The conventional system is a passive system with limited automatic control, where the operator is responsible for decisions and actions that control the process.

4.11.2.6.2 PLC Control Systems (Programmable Logic Controllers)

The PLC based system is a multipurpose system with extensive scope for modification. The plant status, alarms, motor starters, meters and analyzers are all wired into input/output cards located in what are called racks. The racks may be mounted separately or placed in specific plant areas to reduce wiring costs. The input/output racks are associated with controllers that are programmed to perform the required process control functions. Changes can generally be made relatively easily by modification of or addition to the PLC controller programs.

Plant personnel require process information in real-time or in near real-time. The PLC systems accomplish this by means of a Human-Machine-Interface (HMI). The HMI may be dedicated hardware and software or may come in the form of personal computers utilizing HMI software and connected to the PLC communications system. These systems vary widely in their capabilities and performance. The selection of hardware and software should be done carefully to assure current performance and future supportability and expendability.

4.11.3 Design Documents

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

1. Design and construction standards, specifications and installation details;
2. Panel sizing and general arrangements;
3. Control system functional requirements;
4. Control component and instrument data sheets;
5. Operator interface and control hardware and software specifications including input/output lists; and
6. Control system programming and packaged system configuration standards, structure and scope.

4.11.4 Control System Documentation

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

1. Record drawings to show any changes to the design and including any drawings produced during construction;

2. Annotated listings of control system programs and packaged system configuration;
3. Manufacturer's literature for all control and instrumentation components;
4. Final wiring diagrams complete with wire and terminal coding;
5. Motor control schematics;
6. Instrument loop diagrams;
7. Panel wiring and layout details;
8. PLC or DCS wiring schematics;
9. Instrument calibration sheets; and
10. Operating instructions.

4.11.5 Training

Adequate training should be provided to the plant operating and maintenance staff so that the system can be operated to meet the design criteria.

5.1 SCREENING DEVICES

5.1.1 Bar Racks and Screens

5.1.1.1 *Where Required*

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.

5.1.1.2 *Selection Considerations*

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

- effect on downstream treatment and sludge disposal operations;
- possible damage to comminutor or barminutor devices caused by stones or coarse grit particles;
- head losses of the various alternative screening devices;
- maintenance requirements;
- screenings disposal requirements, and quantities of screenings; and
- requirements for a standby unit.

5.1.1.3 *Location*

5.1.1.3.1 *Outdoors*

Screening devices installed outside shall be protected from freezing.

5.1.1.3.2 *Indoors*

Screening devices installed in a building where other equipment or offices are located should be separated from the rest of the building, provided with separate outside entrances and provided with adequate means of ventilation.

5.1.1.3.3 *Access*

Screens located in pits more than 1.2 m deep shall be provided with stairway access. Access ladders are acceptable for pits less than 1.2 m deep, in lieu of stairways.

5.1.1.3.4 *Ventilation*

Fresh air shall be forced into enclosed screening device areas or into open pits more than 1.2 m deep. Dampers should not be used on exhaust or fresh air ducts and fine screens or other obstructions should be avoided to prevent clogging. Where continuous ventilation is required, at least 12 complete air changes per hour shall be provided. Where continuous ventilation would cause excessive heat loss, intermittent ventilation of at least 30 complete air changes per hour shall be provided when workmen enter the area.

Switches for operation of ventilation equipment should be marked and located conveniently. All intermittently operated ventilation equipment shall be interconnected with the respective pit lighting system. The fan wheel should be fabricated from non-sparking material. Gas detectors shall be provided in accordance with Section 4.9.

5.1.1.4 *Design and Installation*

5.1.1.4.1 Bar Spacing

a) 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.

b) 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.

5.1.1.4.2 Velocities

At the design average rate of flow, the screen chamber should be designed to provide a velocity through the screen of approximately 0.3 metres per second to prevent settling, and a maximum velocity during wet weather periods no greater than 0.75 metres per second 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.

5.1.1.4.3 Invert

The screen channel invert should be 75 to 150 mm below the invert of the incoming sewers. To prevent jetting action, the length and/or construction of the screen channel shall be adequate to re-establish hydraulic flow pattern following the drop in elevation.

5.1.1.4.4 Slope

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

5.1.1.4.5 Channels

The channel preceding and following the screen shall be shaped to eliminate stranding and settling of solids and should be designed to provide equal and uniform distribution of flow to the screens. Dual channels shall be provided and equipped with the necessary gates to isolate flow from any screening unit. Provisions shall also be made to facilitate dewatering each unit.

5.1.1.4.6 Flow Measurement

When flow measuring devices need to be in a screen channel, the effect of changes in backwater elevations, due to intermittent cleaning of screens, should be considered in locating of flow measurement equipment. The flow measurement devices should be selected based on reliability and accuracy.

5.1.1.5 *Safety*

5.1.1.5.1 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 be protected by guard railings and deck gratings. Consideration should also be given to temporary access arrangements to facilitate maintenance and repair.

5.1.1.5.2 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.

5.1.1.5.3 Drainage

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

5.1.1.5.4 Lighting

Suitable lighting shall be provided in all work and access areas. Refer to Section 5.1.1.6.2.

5.1.1.6 *Control Systems*

5.1.1.6.1 Timing Devices

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

5.1.1.6.2 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.

5.1.1.6.3 Manual Override

Automatic controls shall be supplemented by a manual override.

5.1.1.7 *Screenings Removal and Disposal*

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

Facilities must be provided for handling, storage, and disposal of screenings in a manner acceptable to the regulatory agency. Separate grinding of screenings and return to the sewage flow is unacceptable.

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.

5.1.1.8 ***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.

5.1.2 Fine Screens

5.1.2.1 ***General***

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 should not be considered equivalent to primary sedimentation. Where fine screens are used, additional provision for the removal of floatable oils and greases shall be considered. Selection of screen capacity should consider flow restriction due to retained solids, gummy material, frequency of cleaning and extent of cleaning.

5.1.2.2 ***Design***

Tests should be conducted to determine BOD_5 and suspended solids removal efficiencies at the design maximum day flow and design maximum day BOD_5 loadings. Pilot testing for an extended time is preferred.

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

Fine screens shall be preceded by a mechanically cleaned bar screen or other protective device. Comminuting devices shall not be used ahead of fine screens. Fine screens shall be protected from freezing and located to facilitate maintenance.

5.1.2.3 ***Electrical Fixtures and Control***

Electrical fixtures and controls in screening areas where hazardous gases may accumulate shall comply with the Canadian Electrical Code and the regulations under the applicable Provincial Power Standards.

5.1.2.4 ***Servicing***

Hosing equipment (with hot and cold water) shall be provided to facilitate cleaning. Provision shall be made for isolating or removing units from their location for servicing.

5.2 COMMINUTORS/GRINDERS

5.2.1 General

Provisions for location shall be in accordance with those for screening devices, Section 5.1.1.3.

5.2.2 When Required

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.

5.2.3 Design Considerations

5.2.3.1 Location

Comminutors or grinders should be located downstream of any grit removal equipment and be protected by a coarse screening device. Consideration for a different sequence may be given to suit individual cases.

5.2.3.2 Size

Comminutor or grinder capacity shall be adequate to handle the design peak hourly flow.

5.2.3.3 Installation

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

Each comminutor or grinder that is not preceded by grit removal equipment should be protected by a 150 mm deep gravel trap.

Gates shall be provided in accordance with Section 5.1.1.4.5.

5.2.3.4 Servicing

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

5.2.3.5 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.

5.3 GRIT REMOVAL FACILITIES

5.3.1 When Required

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.

5.3.2 Location

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

5.3.3 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.

5.3.4 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.

5.3.5 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.

5.3.6 Outside Facilities

Grit removal facilities located outside shall be protected from freezing.

5.3.7 Design Factors

5.3.7.1 Inlet

Inlet turbulence shall be minimized.

5.3.7.2 Type and Number of Units

Grit removal facilities (channel type) should have at least two hand-cleaned units, 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-types are desirable if provided with adequate and flexible controls for agitation and/or air supply devices and with grit collection and removal equipment.

5.3.7.3 Grit Channels

5.3.7.3.1 Velocity

Channel-type chambers shall be designed to provide controlled velocities as close as possible to 0.30 metres per second for normal variation in flow.

5.3.7.3.2 Control Sections

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

5.3.7.3.3 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 fifty (50) per cent allowance for inlet and outlet turbulence.

5.3.7.3.4 Grit Storage

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

5.3.7.4 *Detritus Tanks*

Detritus tanks should be designed with sufficient surface area to remove a 0.2 mm, or smaller, particle with a specific gravity of 2.65 at the expected peak 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:

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

- a classifying-type conveyor can be used to remove the grit and return the organics to the detritus tank;

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

5.3.7.5 *Aerated Grit Tanks*

Aerated grit tanks for the removal of 0.2 mm, or larger, particles with specific gravity of 2.65, should be designed in accordance with the following parameters and as outlined in Table 5.1:

5.3.7.5.1 Air Supply

Air supply should be via air diffusers (wide band diffusion header) positioned lengthwise along one wall of the tank, 600 to 900 mm above the tank bottom. Air supply should be variable. Higher air supply rates should be used with tanks of large cross-section (i.e. greater than 3.6 m deep).

5.3.7.5.2 Inlet Conditions

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

5.3.7.5.3 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.

5.3.7.5.4 Outlet Conditions

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

5.3.7.5.5 Tank Dimensions

The lower limit of the above aeration rates are generally suitable for tanks up to 3.6 m deep and 4.3 m wide. Wider or deeper tanks require aeration rates in the upper end of the 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 are acceptable.

5.3.7.5.6 Velocity

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

5.3.7.5.7 Tank Geometry

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

5.3.7.5.8 Multiple Units

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

TABLE 5.1 – TYPICAL DESIGN INFORMATION FOR AERATED GRIT CHAMBERS[a]

ITEM	VALUE	
	Range	**Typical**
Detention time at peak flowrate, min	2 – 5	3
Dimensions:		
Depth, m	2 – 5	
Length, m	7.5 – 20	
Width, m	2.5 - 7	
Width-depth ratio	1:1 – 5:1	1.5:1
Length-width ratio	3:1 – 5:1	4:1
Air supply, $m^3/min \cdot m$ of length	0.2 – 0.5	
Grit quantities, $m^3/10^3 m^3$	0.004 – 0.200	0.015

a- Metcalf & Eddy Inc., "Wastewater Engineering: Treatment and Reuse", 2003.

5.3.7.6 *Mechanical Grit Chambers*

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

5.3.7.7 *Grit Washing*

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

5.3.7.8 *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.

5.3.7.9 *Water*

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

5.3.8 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.

5.3.9 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.

5.3.10 Grit Disposal

Disposal of grit in sanitary landfills or lagoons, as well as grit incineration shall be considered acceptable disposal methods. Whatever method of disposal is employed, the full spectrum of environmental considerations must be embodied in the final design.

5.4 PRE-AERATION AND FLOCCULATION

5.4.1 General

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

a. Odour control;
b. Grease separation and increased grit removal;
c. Prevention of septicity;
d. Grit separation;
e. Flocculation of solids;
f. Maintenance of DO in primary treatment tanks at low flows;
g. Increased removals of BOD and SS in primary units; and
h. Minimizes solids deposits on side walls and bottom of wetwells.

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.

5.4.2 Arrangement

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

5.4.3 Pre-aeration

5.4.3.1 Air Flow Measurements

Figure 5.1 represents air flow requirements for different periods of pre-aeration.

Pre-aeration periods should be 10 to 15 minutes if odour control and prevention of septicity are the prime objectives.

5.4.4 Flocculation

5.4.4.1 *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.

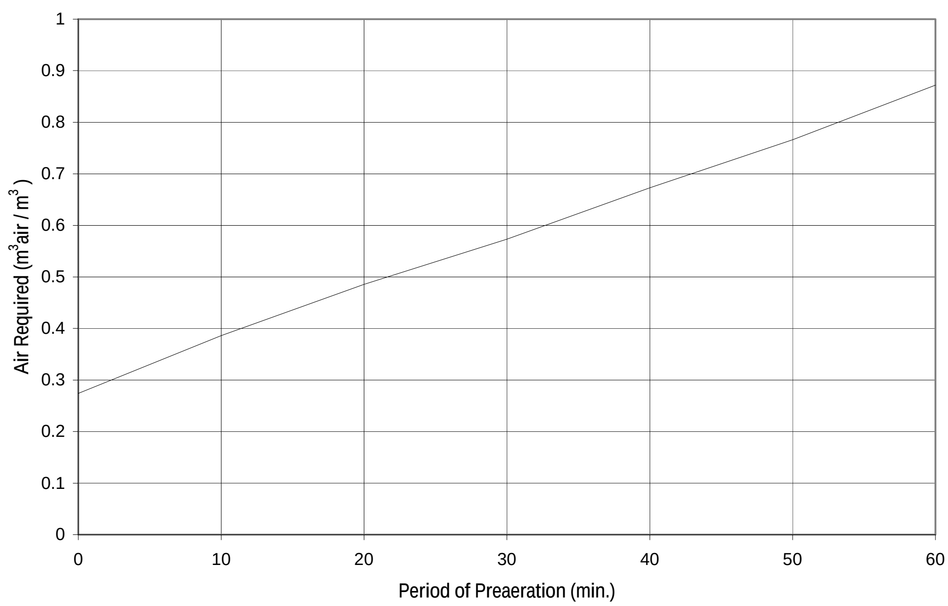
5.4.4.2 *Stirring Devices*

5.4.4.2.1 *Paddles*

Paddles should have a peripheral speed of 0.50 to 0.75 metres per second to prevent deposition of solids.

Figure 5.1

Air Flow Required for Different Periods of Preaeration



5.4.4.2.2 *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.

5.4.4.3 *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.

5.4.4.4 *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, one-half to three minutes.

5.5 FLOW EQUALIZATION

5.5.1 General

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.

5.5.2 Location

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

5.5.3 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.

5.5.4 Size

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

1. Draw a line between the points representing the accumulated volume at the beginning and end of the 24-hr 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.

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 Operation

5.5.5.1 *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.

5.5.5.2 *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 litres per second per cubic metre 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.

5.5.5.3 *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.

5.5.6 Electrical

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

5.5.7 Access

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

6.1 SEDIMENTATION TANKS

6.1.1 General Design Requirements

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

- the characteristics of the raw wastewater; the type of sludge digestion systems, either available or proposed (aerobic digestion should not be used with raw primary sludges):
- the presence, or absence, of secondary treatment following primary treatment;
- the need for handling of waste activated sludge in the primary settling tank:
- the need for, or possible economic benefits through, phosphorus removal in the primary settling tank(s).

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 cubic metres per day. Plants not having multiple units shall include other provisions to assure continuity of treatment.

6.1.1.2 *Arrangement of Units*

Settling tanks shall be arranged in accordance with Section 4.5.10.

6.1.1.3 *Interaction with Other Processes*

a. 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.

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

c. 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.

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.

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.

6.1.1.6 ***Site Constraints***

The selection of feasible clarifier alternatives should include the following site considerations:

a. Wind direction;
b. Proximity to residents;
c. Soil conditions;
d. Groundwater conditions; and
e. Available space.

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.

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 metres per second 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.

6.1.1.9 ***Outlet Arrangements***

6.1.1.9.1 ***General***

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

6.1.1.9.2 ***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.

6.1.1.9.3 ***Weir Troughs***

Weir troughs shall be designed to prevent submergence at maximum design flow and to maintain a velocity of at least 0.3 metres per second at one-half design average flow.

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 to 1 to prevent the accumulation of scum and solids.

6.1.1.11 ***Unit Dewatering***

Unit dewatering features shall conform to the provisions outlined in Section 4.5.3.6 The bypass design should also provide for redistribution of the plant flow to the remaining units.

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.

6.1.1.13 ***Clarifier Covers***

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

6.1.2 Types Of Settling

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.

6.1.2.2 ***Type II Settling (Flocculent Settling)***

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

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

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 flocculent processes. As a result, the contacting particles tend to settle as a zone or blanket, and maintain the same position relative to each other.

6.1.2.4 *Type IV Settling (Compression Settling)*[1]

In Type IV settling, particles have reached such a concentration that a structure is formed and further settling, can only occur by compression. This type of settling typically occurs in the lower layers of a deep sludge mass such as near the bottom of secondary clarifiers and sludge thickeners.

6.1.3 Design Criteria[2]

Table 6.1 and 6.2 display typical design information for primary sedimentation tanks. Table 6.3 displays typical design information for secondary clarifiers for the activated-sludge process. Table 6.4 displays the ranges of overflow rates and BOD and TSS removals from high-rate clarification processes treating wet-weather flows.

TABLE 6.1 TYPICAL DESIGN INFORMATION FOR PRIMARY SEDIMENTATION TANKS

ITEM	UNIT	RANGE	TYPICAL
Primary sedimentation tanks followed by secondary treatment			
Detention time	h	1.5 – 2.5	2.0
Overflow rate			
Average flow	$m^3/m^2 \cdot d$	30 – 50	40
Peak hourly flow	$m^3/m^2 \cdot d$	80 – 120	100
Weir loading	$m^3/m \cdot d$	125 - 500	250
Primary settling with waste activated-sludge return			
Detention time	h	1.5 – 2.5	2.0
Overflow rate			
Average flow	$m^3/m^2 \cdot d$	24 – 32	28
Peak hourly flow	$m^3/m^2 \cdot d$	48 – 70	60
Weir loading	$m^3/m \cdot d$	125 - 500	250

TABLE 6.2 TYPICAL DIMENSIONIAL DATA FOR RECTANGULAR AND CIRCULAR SEDIMENTATION TANKS USED FOR PRIMARY TREATMNT OF WASTEWATER

ITEM	UNIT	RANGE	TYPICAL
Rectangular			
Depth	m	3 – 4.9	4.3
Length	m	15 – 90	24 – 40
Width[a]	m	3 – 24	4.9 – 9.8
Flight Speed	m/min	0.6 – 1.2	0.9
Circular			
Depth	m	3 – 4.9	4.3
Diameter	m	3 – 60	12 – 45
Bottom Slope	mm/mm	1/16 – 1/6	1/12
Flight Speed	r/min	0.02 – 0.05	0.03

a – If widths of rectangular mechanically cleaned tanks are greater than 6 m, multiple bays with individual cleaning equipment may be used, thus permitting tank widths up to 24 m or more.

TABLE 6.3 TYPICAL DESIGN INFORMATION FOR SECONDARY CLARIFIERS FOR THE ACTIVATED-SLUDGE PROCESS					
TYPE OF TREATMENT	**OVERFLOW RATE $m^3/m^2{\cdot}d$**		**SOLIDS LOADING $kg/m^2{\cdot}h$**		**Depth m**
	Average	Peak	Average	Peak	
Settling following air-activated sludge (excluding extended aeration)	16 – 28	40 – 64	4 – 6	8	3.5 - 6
Selectors, biological nutrient removal	16 – 28	40 – 64	5 – 8	9	3.5 – 6
Settling following oxygen-activated sludge	16 – 28	40 – 64	5 – 7	9	3.5 – 6
Settling following extended aeration	8 – 16	24 - 32	1.0 - 5	7	3.5 – 6
Settling for phosphorus removal; effluent concentration, mg/ℓ					3.5 – 6
Total P = 2	24 – 32				
Total P = 1[a]	16 – 24				
Total P = 0.2 to 0.5[b]	12 – 20				

a – Occasional chemical addition required

b – Continuous chemical addition required for effluent polishing

Notes:

1. Peak is a 2-h sustained peak.
2. Weir loading rates are used commonly in the design of clarifiers, although they are less critical in clarifier design than hydraulic overflow rates. Weir loading rates used in large tanks should preferably not exceed 375 m^3/lin m·d of weir at maximum flow when located away from the upturn zone of the density current, or 250 m^3/lin m·d when located within the upturn zone. In small tanks, the weir loading rate should not exceed 125 m^3/lin m·d at average flow or 250 m^3/lin m·d at maximum low. The upflow velocity in the immediate vicinity of the weir should be limited to about 3.5 to 7 m/h.

6.1.4 Sludge And Scum Removal

6.1.4.1 *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.

6.1.4.2 *Sludge Removal*

6.1.4.2.1 *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.

6.1.4.2.2 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.

6.1.4.2.3 Sludge Hopper

The minimum slope of the side walls 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 metres. Extra depth sludge hoppers for sludge thickening are not acceptable. The hoppers are to be accessible for sounding and cleaning.

6.1.4.2.4 Cross-Collectors

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

6.1.4.2.5 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 metre per second 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.

6.1.4.2.6 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 sludges. Sludge pump motor control systems shall include time clocks and valve activators for regulating the duration and sequencing of sludge removal.

6.2 ENHANCED PRIMARY CLARIFICATION

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. Chapter 9 of this manual contains additional information on the selection and application of chemicals for phosphorus removal.

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

6.2.1.1 *Chemical Coagulants*[1]

Chemical coagulants such as ferric chloride and alum (typically < 60 mg/l) provide cations that destabilize colloidal particles in wastewater while flocculent aids such as polymer (typically < 2 mg/l), recycled sludge, and microsand function to accelerate the growth of floc, enlarge the floc, improve floc, shape, strengthen floc structure, and increase particle specific gravity. The use of chemicals allows a higher peak overflow rate during peak events while maintaining or increasing primary clarifier performance, thus minimizing the clarifier surface area that must be provided for peak flows.

Chemically enhanced primary treatment has evolved over time. Early applications typically consisted of simply adding ferric, alum, or lime to a conventionally designed primary settling tank. Only a few plants use lime as a coagulant for primary treatment since lime addition produces more primary sludge because of the chemical solids than do metals salts and lime is more difficult to store, handle, and feed. Coagulant selection for enhanced sedimentation should be based on performance, reliability, and cost. Performance evaluation should use jar tests of the actual wastewater to determine dosages and effectiveness. Operating experience, cost, and other relevant information drawn from other plants should be considered during selection. Current practice uses smaller metal salt doses (20 to 40 mg/ℓ) in combination with polymer addition (< 1mg/l) and includes the use of rapid mix and flocculation before the settling tank. Use of iron salts can decrease the efficiency of downstream disinfection with UV light.

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 overmixing or undermixing. Overmixing may reduce the removal efficiency by breaking up existing wastewater solids and newly formed floc. Undermixing inadequately disperses the chemical, increases chemical use, and reduces the removal efficiency.

The velocity gradient, G, is a measure of mixing intensity. Velocity gradients of 300 s^{-1} are typically sufficient for rapid mix, but some designers have recommended velocity gradients as high as 1,000 s^{-1}. Mechanical mixers, in-line blenders, pumps, baffled compartments, baffled pipes, or air mixers can accomplish rapid mix. The mixing intensity of mechanical mixers and in-line blenders is independent of 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.

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 occurs. Flocculation is slower and more dependent on time and agitation than is the rapid-mix step. Typical detention times for flocculation range between 20 and 30 minutes. Aerated and mechanical grit chambers, flow distribution structures, and influent wells are areas that promote flocculation upstream of primary sedimentation. Advantages and disadvantages of different configurations resemble those for rapid-mix facilities.

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

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.

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.

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 (∅) from the horizontal will have an equivalent settling area which can be calculated utilizing the equation:

$$\text{Total Settling Area} = nA(\cos\varnothing)$$

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.

6.2.2.2 *Configuration*

Typical settling plates are 0.2 m – 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 breakup.

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 $m^3/m^2 \bullet d$.

6.2.4 Dense-Sludge Process[2]

The dense system is a proprietary process and differs from ballasted flocculation in that recycled chemically conditioned solids are used to form microfloc particles with the incoming wastewater entering an air-mixing zone where grit separation occurs and coagulant (usually ferric sulphate) is injected. After mixing, the wastewater flows into the first stage of a two-stage flocculation tank where polymer is added together with chemically conditioned recirculated solids. Recirculated solids accelerate the flocculation process and ensure the formation of dense, homogeneous floc particles. In the second stage of flocculation, grease and scum begin separating and are removed. Flow from the flocculation tank enters a presettling zone and then passes into a plate settler. Most of the suspended flocculated solids are separated directly in the presettling zone; the residual flocculated particles are removed in the settler. A portion of the settled solids in recirculated, and the remainder is sent to the solids processing and disposal system.

TABLE 6.4 RANGES OF OVERFLOW RATES AND BOD AND TSS REMOVALS FROM HIGH-RATE CLARIFICATION PROCESSES TREATING WET-WEATHER FLOWS			
PARAMETER/PROCESS	**BALLASTED FLOCCULATION**	**PLATE AND TUBE SETTLERS**	**DENSE SLUDGE**
Overflow rates			
Low, $m^3/m^2{\cdot}d$	1200 – 2900	880	2300
Medium, $m^3/m^2{\cdot}d$	1800 – 3500	1200	2900
High, $m^3/m^2{\cdot}d$	2300 – 4100	1800	3500
BOD removals, %			
At low overflow rates	35 – 50	45 – 55	25 – 35
At medium overflow rates	40 – 60	35 – 40	40 – 50
At high overflow rates	30 – 60	35 – 40	50 – 60
TSS removals, %			
At low overflow rates	70 – 90	60 – 70	80 – 90
At medium overflow rates	40 – 80	65 – 75	70 – 80
At high overflow rates	30 - 80	40 - 50	70 – 80

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, meat-packing, meat-rendering, and other “oily” wastewaters. Float-solids concentrations are the main performance criteria in solids concentration flotation applications. Concentration applications include the flotation of waste solids of biological, mining, and metallurgical processes.

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 are mixed with the solids produces the best results. Good mixing to ensure chemical dispersion while minimizing shearing forces will provide the best solids-air bubble aggregates.

Numerous factors affect DAF process performance, including:

- Type and characteristics of feed solids;
- Hydraulic loading rate;
- Solids-loading rate;
- Air-to-solids ratio;
- Chemical conditioning;
- Operating policy;
- Float-solids concentration; and
- Effluent clarity.

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.

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.

6.3.1.3 *Solids-Loading Rate*

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

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

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

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 may be required when operating strategy includes maintenance of a specific float-blanket depth or range of depths.

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%.

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.

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.

6.4 PROTECTIVE AND SERVICE FACILITIES

6.4.1 Operator Protection

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

6.4.2 Mechanical Maintenance Access

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

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.

Footnote References

1. WEF, “Manual of Practice FD – 8, Clarifier Design”, 2005
2. Metcalf & Eddy Inc., "Wastewater Engineering: Treatment and Reuse", 2003.

7.1 ACTIVATED SLUDGE

7.1.1 General

7.1.1.1 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.

7.1.1.2 Specific 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 five day BOD. Choice of the process most applicable will be influenced by the proposed plant size, type of waste to be treated, degree and consistency of treatment required, anticipated degree of operation and maintenance and operating and capital costs. All designs shall provide for flexibility in operation.

7.1.1.3 Aeration Equipment Selection

Evaluation of aeration equipment alternatives should include the following considerations:

The size of the aeration tank for any particular modification of the process shall be determined by full scale experience, pilot plant studies, or rational calculations based mainly on food to microorganism ratio or 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.

- Costs - capital, maintenance and operating;
- Oxygen transfer efficiency;
- Mixing capabilities;
- Diffuser clogging problems;
- Air pre-treatment requirements;
- Total power requirements;
- Aerator tip speed of mechanical aerators used with activated sludge systems;
- Icing problems;
- Misting problems; and
- Cooling effects on aeration tank contents.

7.1.1.4 Energy Requirements

This process requires major energy usage to meet aeration demands. Energy costs in relation to critical water quality conditions must be carefully evaluated. Capability of energy usage phase-down while still maintaining process viability, both under normal and emergency energy availability conditions, must be included in the activated sludge design.

7.1.1.5 Winter Protection

Protection against freezing shall be provided to ensure continuity of operation and performance.

7.1.1.6 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.

7.1.1.7 Waste Activated Sludge Concentration

In the absence of sludge thickeners, other effective means of waste sludge concentration shall be provided.

7.1.2 Process Definitions

The following are brief descriptions of a number of modifications of the activated sludge process. See section 7.2 for Sequencing Batch Reactor.

7.1.2.1 Conventional Plug Flow

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.

7.1.2.2 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.

7.1.2.3 Step Feed

Step feed is a modification of the plug flow configuration in which the secondary influent is fed at two or more points along the length of the aeration tank. With this arrangement, oxygen uptake requirements are relatively even, resulting in better utilization of the oxygen supplied. Step feed configurations generally use diffused aeration equipment. Secondary influent flow is usually added in the first 50 to 75% of the aeration tank's length.

7.1.2.4 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.

7.1.2.5 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.

7.1.2.6 Oxidation Ditch

The oxidation ditch is a variation of the extended aeration process. The wastewater is forced around a circular or oval pathway by a mechanical aerator/pumping device at one or more points along the flow pathway. In the aeration tank, the mixed liquor velocity is maintained between 0.2 to 0.37 m/s in the channel to prevent solids from settling.

Oxidation ditches use mechanical brush disk aerators, surface aerators, or jet aerator devices to aerate and propel the liquid flow.

7.1.2.7 High Rate Aeration

This is a type of short-term aeration process in which relatively high concentrations of MLSS are maintained, by utilizing high sludge recirculation rates (100 to 500%), and low hydraulic retention times. 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.

7.1.2.8 High Purity Oxygen

The most common high purity oxygen activated sludge process uses a covered and staged aeration tank configuration. The wastewater, return sludge, and oxygen feed gas enter the first stage of this system and flow concurrently through the tank. The tanks in this system are covered to retain the oxygen gas and permit a high degree of oxygen use. A prime advantage of the staged reactor configuration of the oxygenation system is the system's ability to match approximately the biological uptake rate with the available oxygen gas purity.

7.1.2.9 Countercurrent Aeration[1]

Countercurrent aeration is a unique aeration basin configuration which involves using a circular aeration basin with a centre-pivoted, traveling bridge supporting air diffusers. Rotating aerators continually re-suspend mixed liquor suspended solids while leaving a veil of fine bubbles providing the aeration. Another set of fixed bubble aerators can also be provided. Its rising bubbles are swept along with the rotating liquid current induced by traveling diffusers. The rotating velocity of the liquid causes bubbles from both sources to lead or trail away from their point of release. Clean water efficiencies are typically lower than those of fine-pore disc/dome grid systems and similar to tube-grid arrangements. Because additional energy is required to drive the bridge, it is likely that standard aeration efficiencies will be lower than those for more conventional activated sludge systems.

7.1.3 Return Sludge Equipment

7.1.3.1 Return Activated Sludge Rate

The minimum permissible return sludge rate of withdrawal from the final settling tank is a function of the concentration of 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 tank may be deleterious to both the aeration and sedimentation phases of the activated sludge process, the rate of return activated sludge (RAS) expressed as a percentage of the average design flow of sewage should generally be variable between the limits set forth in the final column of table 7.1.

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.

7.1.3.2 Return Sludge Pumps

If motor driven return sludge pumps are used, the maximum return sludge capacity shall be obtained with the largest pump out of service. A positive head should be provided on pump suctions. Pumps should have at least 75 mm suction and discharge openings.

If air lifts are used for returning sludge from each settling tank hopper, no standby unit will be required, provided the design of the air lifts are such as to facilitate their rapid and easy cleaning and provided other suitable standby measures are provided. Air lifts should be at least 75 mm in diameter.

7.1.3.3 Return Sludge Piping

Suction and discharge piping should be at least 100 mm in diameter and should be designed to maintain a velocity of not less than 0.6 m/s and not more than 2 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 shall be provided.

7.1.3.4 Waste Sludge Facilities

Waste sludge control facilities should be designed for the maximum sludge production of the process. Means for observing, measuring, sampling and controlling waste activated sludge flow shall be provided. Waste sludge may be discharged to the primary settling tank, concentration or thickening tank, sludge digestion tank, mechanical dewatering facilities or any practical combination of these units.

7.1.3.5 Froth Control Units

It is essential to include some means of controlling froth formation in all aeration tanks. A series of spray nozzles may be fixed on top of the aeration tank. Screened effluent or 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.

7.2 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:

a. Fill Method
b. Hydraulic Control Systems
c. Aeration Control Systems
d. Method of Decant
e. Sizing of Disinfection Equipment for Decant Flows
f. Sludge Wasting Methods

One of the main strengths of the SBR process is the process flexibility that can be achieved. Therefore, the above processes can be performed using a variety of methods. Designers of SBR systems must be prepared to supply sufficient detailed information at the request of regulatory authorities.

7.2.1 Process Configurations

One classification of SBR systems distinguishes those that operate with continuous feed and intermittent discharge (CFID) from those that operate with intermittent feed and intermittent discharge (IFID).

7.2.2 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 reactor 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. Stormwater 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 prereaction chamber separated by a baffle wall from the main reaction chamber is also a standard feature of some systems.

7.2.3 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. Some systems distribute the influent over a portion of the reactor bottom so that it will contact settled solids during unaerated and unmixed fill. The end of the fill cycle is controlled either by time (that is, fill for a 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 is stopped, and the mixed liquor is allowed to settle under quiescent conditions (typically 30 to 60 minutes). Next, clarified effluent is decanted until the bottom water level (BWL) is reached. The idle period represents that time 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 stormwater 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.

7.2.4 Sequencing Batch Reactor Equipment

7.2.4.1 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 desk-top computer and software supplied by the PLC vendor. Vendor-developed programs are proprietary and may not be modified by the design engineer or the WWTP operator. Depending on the proprietary software design and type of system, the operator may independently select such variables as solids waste rates; storm cycle times; and aeration, mixing, settling and idle times. In addition, human-machine-interface (HMI) programs are available to present operating data and trends to the operator, to allow the operator to make adjustments in setpoints, to respond to alarm annunciation, and to generate reports. Through high-speed internet or telephone/modem access, the operator is able to monitor and adjust the plant operations remotely.

Programmable logic controller hardware is of modular construction. Troubleshooting procedures are well defined, and replacement of a faulty module is not difficult. An internal battery 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.

7.2.4.2 Reactors

Reactor shapes include rectangular, oval, circular, sloped sidewall, and other unique approaches. Design TWLs and BWLs often allow decanting from 20 to 30% of the reactor contents per cycle.

7.2.4.3 Decanters

Some decanters are mechanically actuated surface skimmers that typically rest above the TWL. The decanter is attached to the discharge pipe by smaller pipes that both support and drain the decanter. The discharge pipe is coupled at each end through seals that allow it to rotate. A screw-type jack attached to a worm gear, sprocket, and chain to an electric motor rotates the decanter from above the TWL to BWL. The speed of rotation is adjustable.

Other decanters are floated on the reactor surface. These decanters may approximate a large-diameter plug valve, whereby the top portion acts as the valve seat (and provides flotation). The bottom is the plug that is connected to a hydraulic operator that moves it away from the seat to allow discharge, or back to the seat to stop discharge. Other floating decanters consist of a length of pipe suspended on floats, with the pipe having a number of orifices bored in the bottom. The number of orifices (and length of pipe) is flow dependent. Each orifice is blocked by a flapper or plugs to prevent solids entry during aeration. There are also decanter configurations that float an effluent discharge pump.

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.

7.2.4.4 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.

7.2.4.5 Aeration/Mixing Systems

A variety of aeration and mixing systems are in use with SBRs. These include jet aeration, fine- and course-bubble aeration, 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.

7.3 ACTIVATED SLUDGE DESIGN PARAMETERS

Table 7.1 shows typical design parameters and efficiencies for various activated sludge process modifications.

TABLE 7.1 – TYPICAL DESIGN PARAMETERS FOR ACTIVATED SLUDGE PROCESS MODIFICATIONS

Process name	Type of Reactor	SRT (days)	F/M (kg BOD/kg MLVSS·d)	Volumetric Loading (kg BOD/m^3/day)	MLSS, mg/ℓ	Detention Time (hrs)	RAS % of influent[b]
High Rate Aeration	Plug Flow	0.5 - 2	1.5 – 2.0	1.2 – 2.4	200 – 1000	1.5 – 3	100 – 150
Contact Stabilization	Plug Flow	5 -10	0.2 - 0.6	1.0 – 1.3	1000 – 3000[c] 6000-10,000[d]	0.5 – 1[c] 2 – 4[d]	50 – 150
High-Purity Oxygen	Plug Flow	1 – 4	0.5 – 1.0	1.3 – 3.2	2000 - 5000	1 - 3	25 – 50
Conventional Plug Flow	Plug Flow	3 - 15	0.2 – 0.4	0.3 – 0.7	1000 - 3000	4 – 8	25 – 75[e]
Step Feed	Plug Flow	3 - 15	0.2 – 0.4	0.7 – 1.0	1500 – 4000	3 – 5	25 - 75
Complete Mix	CMAS	3 - 15	0.2 – 0.6	0.3 – 1.6	1500 – 4000	3 – 5	25 – 100[e]
Extended Aeration	Plug Flow	20 – 40	0.04 – 0.1	0.1 – 0.3	2000 – 5000	20 – 30	50 - 150
Oxidation Ditch	Plug Flow	15 – 30	0.04 – 0.1	0.1 – 0.3	3000 – 5000	15 - 30	75 – 150
Sequencing Batch Reactor, SBR	Batch	10 – 30	0.04 – 0.1	0.1 – 0.3	2000 – 5000[f]	15 – 40	NA
Counter-Current Aeration System	Plug Flow	10 - 30	0.04 – 0.1	0.1 – 0.3	2000 - 4000	15 - 40	25 – 75[e]

a WEF, “Wastewater Treatment Plant Design”, 2003.

b Based on average Flow
c MLSS and detention time in contact basin
d MLSS and detention time in stabilization pond
e For nitrification, rates may be increased by 25 to 50%
f Also used at intermediate SRTs
CMAS = Complete-Mix Activated Sludge.
SRT = Solids Retention Time
MLSS = Mixed Liquor Suspended Solids
MLSS – Mixed Liquor Suspended Solids
RAS – Return Activated Sludge
F/M – Food-to-Microorganism Ratio
MLVSS – Mixed Liquor Volatile Suspended Solids

NA = not applicable

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.

7.4 AERATION

7.4.1 Arrangement of Aeration Tanks

7.4.1.1 General Tank Configuration

a. 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.

Aeration basin depth is an important consideration in the design of aeration systems because of the effect that depth has on the aeration efficiency and air pressure requirements of diffused aeration devices and mixing capabilities of mechanical aerators. A minimum aeration basin depth of 3.0 to 4.6 m is recommended for typical sewage treatment plants. Oxidation ditches should have minimum depth of 1.6 m.

b. Short-circuiting

For very small tanks or tanks with special configuration, the shape of the tank, the location of the influent and sludge return and the installation of aeration equipment should provide for positive control of short-circuiting through the tank.

c. Number of Units

Total aeration tank volume shall be divided among two or more units, capable of independent operation, when the total aeration tank volume required exceeds 140 m^3.

7.4.1.1.1 Inlets and Outlets

a. Controls

Inlets and outlets for each aeration tank unit shall be suitably equipped with valves, gates, stop plates, weirs or other devices to permit controlling the flow to any unit and to maintain a reasonably constant liquid level. The hydraulic properties of the system shall permit the design peak instantaneous hydraulic load to be carried with any single aeration tank unit out of service. The effluent weir for an oxidation ditch must be easily adjustable by mechanical means.

b. Conduits

Channels and pipes carrying liquids with solids in suspension shall be designed to maintain self-cleansing velocities or shall 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.

7.4.1.1.2 Measuring Devices

Devices should be installed for indicating flow rates of raw sewage or primary effluent, return sludge and air to each tank unit. For plants designed for sewage flows of 5000 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 sewage (or primary effluent) at one location, then the mixed liquor flow rate to each aeration unit should be measured.

7.4.1.1.3 Freeboard

All aeration tanks should have a freeboard of not less than 450 mm. Additional freeboard or windbreak may be necessary to protect against freezing or windblown spray. If a mechanical surface aerator is used, the freeboard should not be less than 900 mm.

7.4.1.2 Aeration Equipment

7.4.1.2.1 General

Oxygen requirements generally depend on maximum BOD loading, degree of treatment and level of suspended solids concentration to be maintained in the aeration tank mixed liquor. Aeration equipment shall 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 shall be 1.1 kg O_2/kg peak BOD_5 applied to the aeration tanks with the exception of the extended aeration process, for which the value shall be 1.5. In the case of nitrification, the oxygen requirement for oxidizing ammonia must be added to the above requirement for carbonaceous BOD removal. The nitrogen oxygen demand (NOD) shall be taken as 4.6 times the diurnal peak TKN content of the influent. In addition, the oxygen demands due to recycle flows - heat treatment supernatant, vacuum filtrate, elutriates, etc. - must be considered due to the high concentrations of BOD and TKN associated with such flows.

Careful consideration should be given to maximizing oxygen utilization per unit power input. Unless flow equalization is provided, the aeration system should be designed to match diurnal organic load variation while economizing on power input.

7.4.1.2.2 Variable Oxygenation Capacity

Consideration should be given to reducing power requirements of aeration systems by varying oxygenation capacity to match oxygen demands within the system. Such a system would utilize automatic D.O. probes in each aeration basin to measure dissolved oxygen levels.

7.4.1.2.3 Mixing Requirements

The aeration system which is selected must not only satisfy the oxygen requirements of the mixed liquor, but must also provide sufficient mixing to ensure that the mixed liquor remains in suspension. The power levels necessary to achieve uniform dissolved oxygen and mixed liquor suspended solids concentrations are shown in Table 7.2.

TABLE 7.2 - AERATION MIXING REQUIREMENTS

AERATION SYSTEM	FOR UNIFORM D.O. LEVELS	FOR UNIFORM MLSS LEVELS
MECHANICAL	1.6 TO 2.5 W/cu. m	16 TO 25 W/cu. m
DIFFUSED (COARSE BUBBLE, SPIRAL ROLL)	---	0.33 ℓ/cu. m/s
DIFFUSED (FINE BUBBLE DOMES, FULL FLOOR COVERAGE)	---	0.6 ℓ/sec q. m/s

NOTES:

1. Mixing requirements vary with basin geometry, MLSS concentrations, placement of aeration devices, pumping efficiency of aerators, etc. Wherever possible, refer to full-scale testing results for the particular aerator being considered.
2. ℓ/cu. m/s refers to volume of air per second per volume of aeration tank.
3. ℓ/sec q. m/s refers to volume of air per second per horizontal cross-sectional area of aeration tank.

7.4.1.2.4 Back-up Requirements

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

- effect on the aeration capacity if a piece of equipment breaks down, or requires maintenance (for instance, the breakdown of one of two blowers will have a greater effect on capacity than the breakdown of one of four mechanical aerators);
- time required to perform the necessary repair and maintenance operations;
- the general availability of spare parts and the time required to obtain delivery and installation; and
- means other than duplicate equipment to provide the necessary capacity in the event of a breakdown (for instance, using over-sized mechanical aerators with adjustable weirs to control power draw and oxygenation capacity, or using two speed mechanical aerators, etc.).

Generally considerations such as the above will mean that diffused aeration systems will require a standby blower but mechanical aeration systems may not require standby units, depending upon the number of duty units, availability of replacement parts, etc.

7.4.1.2.5 Oxygen Transfer and Oxygen Transfer Efficiency

Aeration equipment must be designed to carry out its functions under conditions much different than those under which it may be tested by the equipment supplier. The bulk of oxygen transfer tests are conducted under conditions commonly referred to as standard or are corrected to standard conditions.

The designer, therefore, must test the unit under standard conditions and project its efficiency to the mixed liquor, or conduct the test in the mixed liquor. In either case, there are intricacies (related to aerator testing) involved in making this conversion from one condition to another. It is good practice to work with the suppliers when selecting aerators to discuss and agree on, at the time of design, the planned test procedure and interpretation of results.

It is most common to express the oxygenation rate of a particular activated sludge aeration device either as standard oxygen rate, SOR or actual oxygen rate, AOR, both in Kg of oxygen transferred per hour. Either value is considered to be determinable, given the other and its transfer environment. Methods used to calculate oxygen transfer for conditions other than standard, or to correct to standard conditions the results obtained by mixed liquor testing are as follows:

Mechanical surface aerators: Diffused air and submerged turbine aerators:

$AOR = \alpha(SOR)\ (ß(C_{SW}\text{-}C_0)/C_s)\ \Theta^{(T\text{-}20)}$ (1) $AOR = \alpha(SOR)\ (ß(C_{SC^*}\text{-}C_0)/C_s)\ \Theta^{(T\text{-}20)}$ (2)

α = relative rate of oxygen transfer as compared to clean water, dimensionless [equal to 1.0 under standard conditions. Mixed liquor range is 0.6 (near basin influent) to 0.94, with the higher value representative of well-treated wastes; values generally range from 0.8 to 0.94; initial stages of plug flow systems may have very low α; industrial wastes may reduce α];

ß = relative oxygen saturation value as compared to clean water, dimensionless [equal to 1.0 under standard conditions. Mixed liquor range to 0.9 to 0.97, with upper level seen in well-treated wastes];

Θ = temperature correction constant, 1.024;

C_s = oxygen saturation value of clean water at standard conditions, C_s = 9.17 mg/l,

C_{SW} = saturation value of clean water at the surface, at site conditions of temperature, T, and actual barometric pressure, P_a,

C_{SC}^* = corrected C_s value for water depth, D, and oxygen content of gaseous phase, mg/l,

C_0 = initial (or steady state) DO level mg/l [equal to 0.0 mg/l under standard conditions. Mixed liquor values range from 1.5 to 2.0 mg/l at average oxygen uptake conditions], and

T = Temperature of bulk liquid, ℃ [equal to 20℃ under standard conditions. Mixed liquor values range from 5° to 30℃; highest operating temperature is most conservative in terms of design (C_S and $\Theta^{(20-T)}$ to self-correct)].

In the absence of experimentally determined alpha and beta factors, wastewater transfer efficiency shall be assumed to be 50% of clean water efficiency for plants treating primarily (90% or greater) domestic sewage. Treatment plants where the waste contains higher percentages of industrial wastes shall use a correspondingly lower percentage of clean water efficiency and shall have calculations submitted to justify such a percentage.

Oxygen transfer efficiencies are generally represented in terms of kg of O_2 transferred per MJ. Manufacturers of aeration equipment will generally designate the specific equipment O_2 transfer rate as Ns (the standard transfer efficiency or rated capacity). The rated capacity can be expressed as:

$$Ns = \frac{\text{SOR}}{\text{Total Power Consumed (by mixer/aerator, pump, blower)}} \quad (3)$$

The designer must therefore use equation (3) to determine the AOR as described in equation (1) or (2).

7.4.1.2.6 *Characteristics of Aeration Equipment*

Table 7.3 outlines various characteristics of some typical aeration equipment.

7.4.1.2.7 *Diffused Air Systems*

Typical air requirements for all activated sludge processes except extended aeration (assuming equipment capable of transmitting to the mixed liquor the amount of oxygen required in Section 7.4.1.2.1) is 100 cu. metres per kg of BOD_5 peak aeration tank loading. For the extended aeration process the value is 125 m^3.

Air requirements for diffused air systems should be augmented as required by consideration of the following items:

a. To the air requirements calculated shall be added air required for channels, pumps, aerobic digesters or other air-use demand.

b. The specified capacity of blowers or air compressors, particularly centrifugal blowers, should take into account that the air intake temperature may reach 40℃ 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℃ or less and may require oversizing of the motor or a means of reducing the rate of air delivery to prevent overheating or damage to the motor.

c. 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.

d. Diffuser systems shall be capable of providing for the diurnal peak oxygen demand or 200% of the design average day oxygen demand, whichever is larger. The air diffusion piping and diffuser system shall be capable of delivering normal air requirements with minimal friction losses.

TABLE 7.3 - CHARACTERISTICS OF SOME AERATION EQUIPMENT

EQUIPMENT TYPE	EQUIPMENT CHARACTERISTICS	PROCESS WHERE USED	ADVANTAGES	DISADVANTAGES	REPORTED TRANSFER EFFICIENCY* (kg/MJ) FOR STD. CONDITIONS, 0 D0, 20°c, 101 KPa, AND CLEAN WATER
DIFFUSED AIR: A.BUBBLER POROUS DIFFUSERS	PRODUCE FINE-TO-MEDIUM BUBBLES. MADE OF CERAMIC DOMES, PLATES, TUBES, OR PLASTIC-CLOTH TUBE OR BAG.	HIGH RATE, CONVENTIONAL EXTENDED, STEP, MODIFIED, CONTACT-STABILIZATION ACTIVATED SLUDGE PROCESS.	GOOD MIXING; MAINTAINS LIQUID TEMPERATURE. VARYING AIR FLOW PROVIDES GOOD OPERATIONAL FLEXIBILITY.	HIGH INITIAL AND MAINTENANCE COSTS; AIR FILTERS NEEDED; SPIRAL CONFIGURATION LIMITS TANK GEOMETRY.	0.31 - 0.42
NONPOROUS DIFFUSERS	MADE IN BUBBLE CAP, NOZZLE, VALVE, ORIFICE, OR SHEAR TYPES. THEY PRODUCE COARSE OR LARGE BUBBLES. SOME MADE OF PLASTIC WITH CHECK VALVE DESIGN.	SAME AS FOR POROUS DIFFUSERS.	NON-CLOGGING, MAINTAINS LIQUID TEMPERATURE; LOW MAINTENANCE COST.	HIGH INITIAL COST; LOW OXYGEN TRANSFER EFFICIENCY; HIGH POWER COST. FOULING MAY OCCUR.	0.2 - 0.31
B.TUBULAR	PRODUCES HIGH SHEAR AND ENTRAINMENT AS WATER-AIR MIXTURE IS FORCED THROUGH VERTICAL CYLINDER CONTAINING STATIC MIXING ELEMENTS. CYLINDER CONSTRUCTION IS METAL OR PLASTIC.	PRIMARILY AERATED LAGOON APPLICATIONS.	ECONOMICALLY ATTRACTIVE; LOW MAINTENANCE; HIGH TRANSFER EFFICIENCIES FOR DIFFUSED AIR SYSTEMS. WELL SUITED FOR AERATED LAGOON APPLICATIONS.	ABILITY TO ADEQUATELY MIX REACTOR BASIN CONTENTS IS QUESTIONABLE. APPLICATION FOR USE IN HIGH RATE BIOLOGICAL SYSTEMS UNCONFIRMED.	0.31 - 0.44
C. JET	COMPRESSED AIR AND PUMPED LIQUID ARE VIOLENTLY INTERMIXED IN NOZZLE AND AT DISCHARGE INTO VESSEL.	SAME AS FOR BUBBLER DIFFUSER.	SUITED FOR DEEP TANKS; MODERATE COST.	TANK GEOMETRY LIMITED. CLOGGING OF NOZZLE REQUIRES BLOWER AND PUMP. PRIMARY TREATMENT REQUIRED.	0.43 - 0.60
MECHANICAL SURFACE: A.RADIAL FLOW, LOW SPEED 20 - 60 RPM	LOW OUTPUT SPEED; LARGE DIAMETER TURBINE, FLOATING, FIXED-BRIDGE OR PLATFORM MOUNTED. USED WITH GEAR REDUCER.	SAME AS FOR BUBBLER DIFFUSER.	TANK DESIGN FLEXIBILITY; HIGH PUMPING CAPACITY.	SOME ICING IN COLD CLIMATES. INITIAL COST HIGHER THAN AXIAL FLOW AERATORS. GEAR REDUCER MAY CAUSE MAINTENANCE PROBLEMS.	0.34 - 0.76
B.AXIAL FLOW 300 - 1200 RPM	HIGH OUTPUT SPEED. SMALL DIAMETER PROPELLER. THEY ARE DIRECT, MOTOR DRIVEN UNITS MOUNTED ON FLOATING STRUCTURE.	AERATED LAGOONS AND REAERATION.	LOW INITIAL COST; EASY TO ADJUST TO VARYING WATER LEVEL FLEXIBLE OPERATION.	SOME ICING IN COLD CLIMATES; POOR MAINTENANCE ACCESSIBILITY, MIXING CAPACITY MAY BE INADEQUATE.	0.34 - 0.43
C. BRUSH ROTOR	LOW OUTPUT SPEED; USED WITH GEAR REDUCER.	OXIDATION DITCH, APPLIED EITHER AS AN AERATED LAGOON OR AS AN ACTIVATED SLUDGE.	MODERATE INITIAL COST. GOOD MAINTENANCE ACCESSIBILITY.	SUBJECT TO OPERATIONAL VARIABLES WHICH MAY AFFECT EFFICIENCY; TANK GEOMETRY IS LIMITED.	0.43 - 0.60
SUBMERGED TURBINE	UNITS CONTAIN A LOW SPEED TURBINE AND PROVIDE COMPRESSED AIR TO DIFFUSER RINGS OR OPEN PIPE. FIXED-BRIDGE APPLICATION.	SAME AS FOR BUBBLER DIFFUSER.	GOOD MIXING; HIGH CAPACITY INPUT PER UNIT VOLUME; DEEP TANK APPLICATION; OPERATIONAL FLEXIBILITY. NO ICING OR SPLASH.	REQUIRE BOTH GEAR REDUCER AND BLOWER; HIGH TOTAL POWER REQUIREMENTS; HIGH COST.	0.29 - 0.43

* REPORTED EFFICIENCY VARIES BECAUSE OF TANK GEOMETRY, DESIGN AND OTHER FACTORS.

Air piping systems should be designed such that total head loss from the blower outlet (or silencer outlet where used) to the diffuser inlet does not exceed 3.4 kPa at average operating conditions.

The spacing of diffusers should be in accordance with the oxygen requirements through the length of the channel or tank and should be designed to facilitate adjustment of their spacing without major revision to air header piping. Fifty per cent blanks should be provided in at least the first half of the aeration system, for possible addition of more diffusers if found necessary.

All plants shall be designed to incorporate removable diffusers that can be serviced and/or replaced without dewatering the tank.

e. Individual assembly units of diffusers shall 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.

f. 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. Blowers must have silencers, flexible connections and gauges.

g. Blowers which require internal lubrication are not desirable because of the danger of diffuser clogging from oil being carried in the air stream. Water-piston-type compressors are not desirable because they increase the condensation in the air system, resulting in a more severe corrosion problem in the piping and greater pressure loss required to pass the condensate through the diffusers.

7.4.1.2.8 Mechanical Aeration Systems

a. 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. In the absence of specific design information, the oxygen requirements shall be calculated using a transfer rate not to exceed 1.22 kg 0_2/kW·h in clean water under standard conditions.

b. Design Requirements

1. maintain a minimum of 2.0 mg/l dissolved oxygen in the mixed liquor at all times throughout the tank or basin;
2. maintain all biological solids in suspension;
3. meet maximum oxygen demand and maintain process performance with the largest unit out of service; and

4. provide for varying the amount of oxygen transferred in proportion to the load demand on the plant.

5. provide that motors, gear housing, bearings, grease fittings, etc., be easily accessible and protected from inundation and spray as necessary for the proper functioning of the unit.

c. Winter Protection

Due to high heat loss, the mechanism, as well as subsequent treatment units, shall be protected from freezing where extended cold weather conditions occur.

7.5 ROTATING BIOLOGICAL CONTACTORS

7.5.1 General

7.5.1.1 Applicability

The Rotating Biological Contactor (RBC) process may be used where sewage 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:

- Raw sewage amenability to biological treatment;
- Pretreatment effectiveness including scum and grease removal;
- Expected organic loadings, including variations;
- Expected hydraulic loadings, including variations;
- Treatment requirements, including necessary reduction of carbonaceous and/or nitrogenous oxygen demand;
- Sewage characteristics, including pH, temperature, toxicity, nutrients;
- Maximum organic loading rate of active disc surface area;
- Minimum detention time at maximum design flow.

7.5.1.2 Winter Protection (Enclosures)

Wastewater temperature affects rotating contactor performance. Year-round operation requires that rotating contactors be covered to protect the biological growth from cold temperatures and the excessive loss of heat from the wastewater with the resulting loss of performance.

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.

7.5.1.3 Required 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 or grit, debris and excessive oil or grease prior to the RBC units. Bar screening or comminution are not sole means of pretreatment.

7.5.1.4 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.

7.5.1.5 Operating Temperature

The temperature of wastewater entering any RBC should not drop below 13℃ 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.

7.5.1.6 Design Flexibility

Adequate flexibility in process operation should be provided by considering one or more of the following:

a) Variable rotational speeds in first and second stages;
b) Multiple treatment trains;
c) Removable baffles between all stages;
d) Positive influent flow control to each unit or flow train;
e) Positively controlled alternate flow distribution systems;
f) Positive airflow metering and control to each shaft when supplemental operation or air drive units are used;
g) Recirculation of secondary clarifier effluent.

7.5.1.7 Hydrogen Sulphide

When higher than normal influent or sidestream hydrogen sulphide concentrations are anticipated, appropriate modifications in the design should be made.

7.5.2 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.

7.5.2.1 Unit Sizing Considerations

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:

a) design flow rate and influent waste strength;
b) percentage of BOD to be removed;
c) media arrangement, including number of stages and unit area in each stage;
d) rotational velocity of the media;
e) retention time within the tank containing the media;
f) wastewater temperature; and
g) 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 allowable effluent ammonia nitrogen concentration.

7.5.2.2 *Hydraulic Loading*

Hydraulic loading to the RBC's should range between 75 to 155 l/m^2d of media surface area without nitrification, and 30 to 80 l/m^2d with nitrification.

7.5.2.3 *Organic Loading*

The RBC process is approximately first order with respect to BOD removal; ie., 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.

7.5.2.4 *Tank Volume*

For purposes of plant design, the optimum tank volume is measured as wastewater volume held within a tank containing a shaft of media per unit of growth covered surface on the shaft, or litres per square metre (l/m^2). The optimum tank volume determined when treating domestic wastewater up to 300 mg/l BOD is 4.9 l/m^2, which takes into account wastewater displaced by the media and attached biomass. The use of tank volumes in excess of 4.9 l/m^2 does not yield corresponding increases in treatment capacity when treating wastewater in this concentration range.

7.5.2.5 *Detention Time*

Based on a tank volume of 4.9 l/m^2, the detention time in each RBC stage should range between 40 to 120 minutes without nitrification, and 90 to 250 minutes with nitrification.

7.5.2.6 Media Submergence and Clearance

RBC's should operate at a submergence of approximately 40 percent based on total media surface area. To avoid possible shaft overstressing and inadequate media wetting, the liquid operating level should never drop below 35 percent submergence. Media submergence of up to 95 percent 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.

7.5.3 Design Considerations

7.5.3.1 Unit Staging

The arrangement of media in a series of stages has been shown to significantly increase treatment efficiency. It is therefore recommended that an RBC plant be constructed in at least four stages for each flow path (or four zones of media area).

Four stages may be provided on a single unit by providing baffles within the tank. For small installations where the total area requirements dictate two units per flow path, two units may be placed in series with a single baffle in each tank, thus providing the minimum of four stages. For larger installations requiring four or more units per flow path, the units may be placed in a series within the flow path, with each unit itself serving as a single stage. Generally, though, plants requiring more than four stages should be constructed in a series of parallel floor trains, each comprised of four separate stages.

Wastewater flow to RBC units may be either perpendicular or parallel to the media shafts.

7.5.3.2 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.

7.5.3.3 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.

7.5.3.4 Shaft Rotational Speed

The shafts are rotated (1 to 2 revolutions per minute) by either mechanical or compressed air drive. Provision should also be made for rotational speed control and reversal.

7.5.3.5 Biomass Removal

A means for removing excess biofilm growth should be provided, such as air or water stripping, chemical additives, etc.

7.5.3.6 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.

7.5.3.7 Supplemental Air

Periodic high organic loadings may require supplemental aeration in the first stage to promote sloughing of biomass.

7.5.3.8 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.

7.5.3.9 Recirculation

Consideration should be given to providing recirculation 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.

7.5.3.10 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 underloaded 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.

7.5.3.11 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.

7.5.3.12 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.

7.5.3.13 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 manufacturers' 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.

7.5.3.14 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.

7.6 WASTE STABILIZATION PONDS

7.6.1 Supplement to Pre-Design Report

7.6.1.1 General

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 information must be submitted in addition to that required in Section 1.3.

7.6.1.2 Location in Relation to Nearby Facilities

The location and direction of all residences, commercial developments, recreational areas and water supplies within two kilometres of the proposed pond shall be included in the Pre-Design report.

7.6.1.3 Land Use Zoning

Land use zoning adjacent to the proposed pond site shall be included.

7.6.1.4 Soil Borings

Data from soil borings, conducted by an independent soil testing laboratory to determine subsurface soil characteristics and groundwater characteristics (including elevation and flow) of the proposed site and their effect on the construction and operation of a pond, shall also be provided. At least one boring shall be a minimum of 7.5 m in depth or into bedrock, whichever is shallower. If bedrock is encountered, rock type, structure and corresponding geological formation data should be provided. The boring shall be filled and sealed. The permeability characteristics of the pond bottom and pond seal materials shall also be studied.

7.6.1.5 Percolation Rates

Data demonstrating anticipated percolation rates at the elevation of the proposed pond bottom shall be included. Insitu permeability testing should be done to measure the percolation rates.

7.6.1.6 Site Description

A description, including maps showing elevations and contours of the site and adjacent area suitable for expansion shall be identified. Due consideration shall be given to additional treatment units and/or increased waste loadings or determining load requirements.

7.6.1.7 Location of Field Tile

The location, depth, and discharge point of any field tile (subsurface drainage systems) in the immediate area of the proposed site shall be identified so that proper separation distances from proposed facilities can be maintained.

7.6.1.8 Sulphate Content of Water Supply

Sulphate content of the basic water supply shall be determined.

7.6.1.9 Well Survey

A pre-construction survey of all nearby wells (water level and water quality) is mandatory.

7.6.2 Location

7.6.2.1 Distance From Habitation

For separation distances, see Section 4.3.2.

7.6.2.2 Prevailing Winds

If practicable, ponds should be located so that local prevailing winds will be in the direction of uninhabited areas.

7.6.2.3 Surface Runoff

Location of ponds in watersheds receiving significant amounts of storm water runoff is discouraged. Adequate provision must be made to divert storm water runoff around the ponds and protect pond embankments from erosion.

7.6.2.4 Groundwater Pollution

Existing wells which serve as drinking water sources should be protected from health hazards or as required by the regulatory agency having jurisdiction. Possible travel of pollutants through porous soils and fissured rocks should be objectively evaluated to safeguard the wells. A pond 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 1.2m between the bottom of the pond and the maximum groundwater elevation should be maintained; however, less separation may be acceptable when supported by appropriate hydrogeological and engineering designs/investigations upon acceptance of the regulatory authority having jurisdiction.

A minimum separation of 1.5m between the bottom of the pond and bedrock is recommended; however, less separation may be acceptable when supported by appropriate hydrogeological and engineering designs/investigations upon acceptance of the regulatory authority having jurisdiction.

7.6.2.5 Protection of Surface Water Supplies

Stabilization basins shall be located downhill, downstream and remote from all sources of surface water supplies (lakes and rivers). The following minimum distances shall be employed as the criteria:

Minimum Distance from a Lake or River to the Centre of a Dyke of a Proposed Stabilization Basin	Remarks
120 m	Lined stabilization basin, pervious soil
75 m	Lined stabilization basin, impervious soil

7.6.2.6 *Geology*

Ponds 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).

7.6.2.7 *Floodplains*

A pond shall not be located within the 100 year floodplain.

Design considerations should be made to minimize damages from the impacts of storm surges in coastal areas.

7.6.3 Definitions[1]

7.6.3.1 *Aerobic Stabilization Basin*

Aerobic lagoons are shallow basins which use natural processes involving both algae and bacteria. Aerobic lagoons have a minimum depth of 1 m. Oxygen is provided by algae during photosynthesis and wind aided surface aeration. The physical dimensions, temperature, amount of sunlight, and amount of natural or artificial turbulence are used to maintain a desired dissolved oxygen concentration. In practice it is not possible to maintain a completely aerobic lagoon. The bottom sediments will contain some facultative bacteria.

7.6.3.2 *Facultative Stabilization Basins*

Facultative lagoons are the most common type and are also referred to as oxidation lagoons. These lagoons are typically 1.5 m deep, with detention times ranging from 25 to more then 180 days. Depths are kept at 1.5 m or more to avoid the growth of emergent plants. Surface layers of the lagoon are aerobic with an aerobic layer near the bottom. Oxygen is supplied by surface aeration and photosynthetic algae. Facultative lagoons are designed in series with a minimum of three cells to reduce short circuiting. The primary problem with facultative lagoons is the production of algae that remains in the effluent, which sometimes causes effluent suspended solids to exceed discharge requirements.

7.6.3.3 *Aerated Stabilization Basins*

Aerated lagoons can be either partially mixed or completely mixed. Oxygen is supplied by mechanical floating aerators or diffused aeration. Aerated lagoons have a minimum depth of 3 m, with detention times ranging from 5 to 30 days. Aerated lagoons accept higher biochemical oxygen demand (BOD) loadings than facultative lagoons, are less susceptible to odours, and typically require less land. Aerated lagoons are followed by a facultative lagoon or a settling lagoon (1-day detention or less) to reduce suspended solids before discharge.

7.6.3.4 *Anaerobic Stabilization Basins*

Anaerobic lagoons are heavily loaded with organics and do not have an aerobic zone. They have a minimum depth of 3 m and detention times of 20 to 50 days.

Biological activity is typically low when compared to that of a mixed anaerobic digester. Anaerobic lagoons have been used as pretreatment to facultative and aerobic lagoons for strong industrial wastewater and for rural communities with a significant organic load from industries such as food processing.

7.6.4 Application, Advantages and Disadvantages of Different Stabilization Basin Types

Table 7.4 presents advantages and disadvantages of the different pond and stabilization basin types for various applications.

TABLE 7.4 - APPLICATION, ADVANTAGES AND DISADVANTAGES OF THE DIFFERENT STABILIZATION BASIN TYPES

PARAMETER	UNAERATED AEROBIC	FACULTATIVE	ANAEROBIC	AERATED	
				AEROBIC	FACULTATIVE
APPLICATION	NUTRIENT REMOVAL; TREATMENT OF SOLUBLE ORGANIC WASTES; SECONDARY EFFLUENTS	TREATMENT OF RAW DOMESTIC AND INDUSTRIAL WASTES	PRETREATMENT FOR STRONG INDUSTRIAL WASTEWATER AND AREAS WITH LARGE ORGANIC LOAD	TREATMENT OF RAW DOMESTIC AND INDUSTRIAL WASTES	TREATMENT OF RAW DOMESTIC AND INDUSTRIAL WASTES
ADVANTAGES	LOW OPERATING AND MAINTENANCE COSTS	LOW OPERATING AND MAINTENANCE COSTS;	SMALL VOLUME AND AREA; EFFECTIVE FOR HIGH STRENGTH WASTEWATER	SMALL VOLUME AND AREA; RESISTANCE TO UPSETS	SMALL VOLUME AND AREA; RESISTANCE TO UPSETS
DISADVANTAGES	LARGE VOLUME AND AREA; POSSIBLE ODOURS	LARGE VOLUME AND AREA; POSSIBLE ODOURS	SIGNIGICANT MAINTENANCE; GAS SAFETY; POSSIBLE ODOURS	SIGNIFICANT MAINTENANCE AND OPERATING COSTS; HIGH SOLIDS IN EFFLUENT; FOAMING	MAINTENANCE AND OPERATION COSTS; FOAMING

7.6.5 Basis of Design

7.6.5.1 Waste Stabilization Basins

7.6.5.1.1 Holding Capacity Requirements

Before the design of a waste stabilization pond system can be initiated, the designer shall determine the following:

- whether the stabilization basin can be continuously discharged or operate on a fill-and-draw basis (Intermittent Discharge);
- the period of the year if any, when discharge will not be permitted;
- what discharge rates will be permitted with fill-and-draw stabilization basin and what, if any, provision must be made for controlling effluent discharge rates in proportion to receiving stream flow rates;
- what the minimum time for discharge of stabilization basin cell contents

should be for fill-and-draw systems.

The holding capacity of ponds shall be based upon average daily sewage flow rates, making a special allowance for net precipitation entering the cells.

7.6.5.1.2 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 regulatory authority having jurisdiction.

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 winter-time flows.

Design variables such as pond depth, multiple units, detention time and additional treatment units must be considered with respect to applicable standards for BOD_5, total suspended solids (TSS), fecal coliforms, dissolved oxygen (DO) and pH.

7.6.5.1.3 Flow Distribution

The main inlet sewer or forcemain should terminate at a chamber which permits hydraulic and organic load splitting between the stabilization basin 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.

7.6.5.1.4 Controlled -Discharge Stabilization Ponds

For controlled-discharge systems, the area specified as the primary ponds should be equally divided into two cells. The third or secondary cell volume should, as a minimum, be equal to the volume of each of the primary cells.

In addition, the design should permit for adequate elevation difference between primary and secondary ponds to permit gravity filling of the secondary from the primary. Where this is not feasible, pumping facilities may be provided.

7.6.5.1.5 Flow-Through Pond Systems

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 two metres to facilitate both solids and pathogen reduction.

7.6.5.1.6 Tertiary Pond

When ponds are used to provide additional treatment for effluents from existing or new secondary sewage treatment works, the reviewing authority will, upon request, establish BOD loadings for the pond after due consideration of the efficiencies of the preceding treatment units.

7.6.5.2 Aerated Stabilization Basins

7.6.5.2.1 General

Aerated ponds can be either aerobic or facultative. An aerated aerobic pond contains dissolved oxygen through the whole system with no anaerobic zones. The pond shape and the aerating power provides complete mixing. The aerated facultative pond provides a partially mixed condition which will cause an anaerobic zone to develop at the bottom as suspended solids settle due to low velocity in the system.

a. Aerated Aerobic Stabilization Basins

In general, an aerated pond can be classified as an aerobic pond (complete mixed) if the mechanical aeration power level is above six watts per cubic metres of maximum storage.

Aerated aerobic ponds should be designed to maintain complete mixing with bottom velocities of at least 0.15 m/s. It is important that sufficient mixing power be provided.

Quiescent settling areas adjacent to the aerated cell outlets or the addition of 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 one metre 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.

b. Aerated Facultative Stabilization Basins

Aerated facultative ponds should be designed to maintain a minimum of two 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 kilogram of oxygen per kilogram of BOD_5 applied uniformly throughout the pond when the water temperature is 20℃. The organic loading rate should be maintained between 0.031 and 0.048 $kg/m^3 \cdot day$.

The escape of algae into the effluent should be controlled by providing a quiescent area adjacent to each cell outlet with an overflow rate of 32 $m^3/m^2 \cdot d$. If multiple aerated facultative cells are used, all cells following the first one shall have diminished aeration capacity to permit additional settling.

Whenever possible, provisions should be provided for recirculating part (5-10%) of the final aeration cell effluent back into the influent in order to maintain a satisfactory mix of active micro-organisms.

7.6.5.2.2 *Design Approach*

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

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

7.6.5.2.2.1 *CBOD Removal and Effluent Characteristic*

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

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

Where:

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

The selection of the reaction rate coefficient is critical in the design of the lagoon system. All other considerations in the design will be influenced by this selection. If possible, a design K_{20} should be determined for the wastewater in pilot or bench scale tests; experiences of others with similar wastewaters and environmental conditions should also be evaluated. Reaction rate coefficient K_{20} may vary from 0.276 day^{-1} for complete mix cell to 0.138 day^{-1} for aerated cell.

When using the complete mix model, the number of cells in series has a pronounced effect on the size of the aerated cell required to achieve a specific degree of treatment. The reactor required to achieve a given efficiency may be greatly reduced by increasing the number of cells in series.

7.6.5.2.2.2 *Temperature Effects*

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

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

where:

K_t = Reaction rate coefficient at t℃, day $^{-1}$

K_{20} = Reaction rate coefficient at 20℃, day $^{-1}$
t = Wastewater temperature, ℃
θ = Temperature activity coefficient (varies from 1.04 to 1.1 for aerated lagoons, with typical value of 1.035)

7.6.5.2.2.3 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 two mg/l in the ponds at all times.

The oxygen requirements should meet or exceed the peak 24 hours summer loadings. A safety factor of up to two should be considered in designing oxygen supply equipment based on average BOD_5 loadings. The amount of oxygen requirement has been found to vary from 0.7 to 1.5 times the amount of BOD_5 removed. Suitable protection from weather shall be provided for electrical control.

7.6.5.2.2.4 Mixing Requirements

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

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

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

7.6.5.2.2.5 Solids Separation

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

7.6.5.3 Industrial Wastes

Due 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.

7.6.5.4 Multiple Units

At a minimum, a waste stabilization pond system shall consist of two cells designed to facilitate both series and parallel operations. The maximum size of a pond cell should be five hectares. A one cell system may be utilized in small installations. Larger cells may be permitted for bigger installations.

All systems should be designed with piping flexibility to permit isolation of any cell without affecting the transfer and discharge capabilities of the total system.

Requirements for multiple units in an aerated stabilization basin system shall be similar to those in an activated sludge system, including requirements for back-up aeration equipment.

7.6.5.5 Design Depth

The minimum operating depth should be sufficient to prevent growth of aquatic plants and damage to the dykes, control structures, aeration equipment and other appurtenances. See Section 7.6.3 for typical pond depths.

7.6.5.6 Pond Shape

Acute angles within any wastewater stabilization pond or aerated stabilization basin should be avoided. Square cells are preferred to long narrow rectangular cells. The long dimension of any pond should not align with the prevailing wind direction.

7.6.6 Pond Construction Details

7.6.6.1 Embankments and Dykes

7.6.6.1.1 Materials

Embankments and dykes shall be constructed of relatively impervious materials and compacted to at least 95 percent Standard Proctor Density to form a stable structure. Vegetation and other unsuitable materials should be removed from the area where the embankment is to be placed.

A soils consultant's report shall be required for all earthen berm construction to demonstrate the suitability of the soils. In certain instances a hydrogeologist's report may be required to assess possible impact on the water table. All topsoil must be stripped from the area on which the berms are to be constructed.

7.6.6.1.2 Top Width

The minimum embankment top width should be three metres to permit access of maintenance vehicles.

7.6.6.1.3 Maximum Slopes

Unless otherwise specified by a soil consultant's report, embankment slopes should not be steeper than:

a. Inner

Three horizontal to one vertical.

b. Outer

Three horizontal to one vertical.

7.6.6.1.4 Minimum Slopes

Embankment slopes should not be flatter than:

a. Inner

Four horizontal to one vertical. Flatter slopes are sometimes specified for larger installations because of wave action but have the disadvantage of added shallow areas conducive to emergent vegetation. Other methods of controlling wave action may be considered.

b. Outer

Outer slopes shall be sufficient to prevent surface water runoff from entering the ponds.

7.6.6.1.5 Freeboard

Minimum freeboard shall be one metre.

7.6.6.2 Erosion Control

a. 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 rip rap 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.

b. 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 rip rap 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.

c. 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 7.6.6.2 b.

d. Additional Erosion Protection

Rip rap or some other acceptable method of erosion control is required as a minimum around all piping entrances and exits. For aerated cells the design should ensure erosion protection on the slopes and bottoms in the areas where turbulence will occur.

e. Erosion Control During Construction

Effective site erosion control shall be provided during construction according to applicable provincial documents such as "Erosion and Sedimentation Control Handbooks for Construction Sites", if available in the province of jurisdiction. An approved erosion control plan is required before construction begins.

7.6.6.3 Vegetation Control

A method shall be specified which will prevent vegetation growth over the surface of the inner slope and top of the embankment.

7.6.6.4 Pond Bottom

7.6.6.4.1 Vertical Separation

For separation distance between the cell bottom and bedrock, refer to Section 7.6.2.4. Cell bottoms should be located sufficiently high above the groundwater level, in order to prevent inflow and/or liner damage.

7.6.6.5 Uniformity

The pond bottom should be as level as possible at all points. Finished elevations should not be more than 75 mm from the average elevation of the bottom.

7.6.6.5.1 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 Design Section 7.6.6.2 a.

7.6.6.5.2 Permeability Tests

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.

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 one metre with carefully tamped soil. Particular care should be given to tamping the 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 3 twenty-four hour periods, or until the readings become consistent. (Level shall be re-adjusted to 1.2 m at the beginning of each 24 hour period).

7.6.6.5.3 Interpretation of Hydraulic Conductivity Measurements

There can be major differences between laboratory and field hydraulic conductivity measurements. These differences are likely to occur because of complex geological and hydrogeological conditions, in situ and errors in measurement methods. The ratio of K (in situ) to K (laboratory) may be in the range of 0.38 to 64. The major reasons for higher field values are: 1) laboratory tests are generally run on homogeneous, clayey samples; 2) sand seams, fissures and other macrostructures in the field are not present in laboratory samples; 3) measurement of vertical K in the laboratory and horizontal K in the field; and 4) changes in soil structure, chemical characteristics of the permeant, air entrapment in laboratory samples and other errors associated with laboratory tests.

The value of K from a field test as described above may be obtained from the following equation:

$$K = (A/FDt) \times [\ln(h_1/h_2)]$$

Where,

A = area of standpipe

t = time for head change from h_1 to h_2

D = diameter of hole

h = head water above water table

F = 2.0 for a borehole with a flat bottom at an upper impervious boundary,

or

= 2.75 for a cased borehole with a flat bottom in the middle of a deep soil layer.

* Olson and Daniel (1981)

Alternatively, if the laboratory tests show a permeability greater than 1 x 10^{-6} cm per second, or if the drop in head of the field test exceeds 10 mm per 24 hours, then provision should be made to make the soil more impermeable, as indicated in Design Section 7.6.6.5.6.

7.6.6.5.4 Soil

Soil used in constructing the pond bottom (not including liner) and dyke cores shall be relatively incompressible and tight and compacted at or up to four percent above the optimum water content to at least 90 percent Standard Proctor Density. Soft pockets that would prevent sufficient compaction of the liner must be sub-excavated and replaced with suitable, compacted fill.

7.6.6.5.5 Liner

Stabilization Ponds 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.

7.6.6.5.6 Seepage Control Criterion for Clay Liners

The seepage control criterion for municipal wastewater stabilization ponds and aerated stabilization basins utilizing clay liners specifies a maximum hydraulic conductivity, K, for the pond liner as a function of the liner thickness, L, and water depth, D, by the equation:

$$\text{Maximum K (m/s)} = \frac{4.6 \times 10^{-8}\,\text{m/s} \times \text{L (m)}}{\text{D (m)} + \text{L (m)}}$$

where all units are in metres and seconds.

For example, a compacted clay liner that is 0.5 m thick must have a hydraulic conductivity of about 1.3 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 metres per day per hectare at a water depth of 1.2 metres.

7.6.6.6 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.

7.6.6.7 Site Drainage

Surface drainage must be routed around and away from cells. Field tiles within the area enclosed by the berms must be located and blocked so as to prevent cell content leakage. Measures must be taken, where necessary, to avoid disruption of field tile and surface drainage of adjacent lands, by constructing drainage works to carry water around the site.

7.6.7 Design and Construction Procedures for Clay Liners

7.6.7.1 Delineation of Borrow Deposit

The first step in designing a compacted clay liner is delineating a relatively uniform deposit of suitable borrow material, preferably from the pond cut or from a nearby borrow area. The required volume of clayey soil is equal to the surface area of the pond interior times the liner thickness (measured perpendicular to the bottom and side slope surfaces). A large reserve volume is recommended to ensure that there is indeed sufficient clay volume after removing silt and sand pockets and other unsuitable materials.

7.6.7.2 Liner Thickness

Recommended minimum compacted clay liner thicknesses are 0.5 m on the pond bottom and 0.7 m on the side slopes, to allow for weathering, variations in actual thickness, pockets of poor quality material that escape detection, etc. If a clay core in the dyke is preferred over an upstream clay blanket liner, then the core should be well keyed into the bottom liner. A minimum core width of three metres is suggested to allow economic and proper placement and compaction of the clay using large earth-moving equipment.

7.6.7.3 Hydraulic Conductivity of Compacted Clay

The in situ hydraulic conductivity of the compacted clay liner should be predicted from laboratory tests on the proposed clay borrow material. Several samples should be selected representing the range of material within the designated borrow zone, not just the better material. Permeability tests should be performed on the samples compacted to the required density (i.e. 95% of standard Proctor maximum dry density) at a moisture content anticipated in the field. It is recommended that the sensitivity of the compacted clay hydraulic conductivity to variations in density and moisture content be determined. The designer must be prepared to ensure that the soil is brought to the specified moisture content (i.e. by wetting), unless the natural moisture content is already suitable.

A laboratory value for K should be calculated from the weighted average of the individual tests. The weighting of each test value should be according to the estimated percent of the borrow volume that the individual sample represents.

It is recommended that the liner design be based on a K in situ that is one order of magnitude larger than the average K (lab), i.e.:

$$K\ (\text{design}) = K\ (\text{in situ}) = 10 \times \text{average } K\ (\text{lab})$$

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 7.6.6.5.8. 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.

7.6.7.4 Subgrade Preparation

Clay should not be placed directly over gravel or other materials that do not provide an adequate filter to prevent piping erosion of the liner.

7.6.7.5 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.

7.6.7.6 Construction Control

The most important form of quality control during construction of compacted clay liners will be observation and direction by the engineer. The characteristics of the desired liner material should be established in as much detail as possible (i.e. by colour, texture, moisture content, plasticity or characteristic features such as the mineralogy of pebbles in till). Quick visual or index test identification by experienced field personnel is probably the best way to detect poor quality material. An indirect but simple way of controlling liner quality is to perform frequent in situ density and moisture content tests. The density and moisture content may then be related to hydraulic conductivity by the relationships established during the laboratory test program (see Section 7.6.7.3). The frequency of tests should be increased when soil conditions are variable. The tests may be used to statistically evaluate the overall liner properties and to assess suspect zones in the liner.

In situ density and moisture content tests should be carried out on a routine basis for each lift. Tests should be conducted on a grid pattern (say 30 x 30 m to 60 x 60 m grids for large ponds and at closer spacing for small ponds) and in suspect areas.

The completed liner may be assessed by performing in situ infiltration tests, which may be theoretically related to hydraulic conductivity values (see Section 7.6.6.5.3). It should be noted that the compacted clay liner is most likely to be partially saturated at the end of construction. The presence of five to ten percent 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.

7.6.7.7 Planning

The most important aspect of constructing a compacted clay liner may be the planning stage when the inspection engineer's role is defined, contract specifications are prepared and construction strategies are worked out. The engineer must have an adequate degree of control over material selection and methods of placement. The work procedure must be flexible with respect to earth movement.

Ideally, the borrow for a compacted clay liner would be the cut material just below the eventual pond invert. Thus, material may be cut and placed in a single operation for much of the pond liner area, although some stockpiling of borrow may be inevitable.

The lower lift of the liner might consist of reworked native soil broken up by tilling and re-compacted to eliminate fissures, etc. Nevertheless, the contract should allow for selective borrowing of cut material for liner use, for stockpiling, removal of undesirable materials and possible additional borrowing outside of the cut area.

7.6.8 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 Design Section 7.6.6.2 b before the introduction of water.

7.6.9 Influent Lines

7.6.9.1 Material

Any generally accepted material for underground sewer construction will be given consideration for the influent line to the pond. Unlined corrugated metal pipe should be avoided however, due to corrosion problems. In material selection, consideration must be given to the quality of the wastes, exceptionally heavy external loadings, abrasion, soft foundations and similar problems.

7.6.9.2 Surcharging

The design and construction of influent piping shall insure that where surcharging exists, due to the head of the pond, no adverse effects will result. These effects shall include basement flooding and overtopping of manholes.

7.6.9.3 Forcemains

Forcemains terminating in a sewage stabilization basin should be fitted with a valve immediately upstream of the stabilization basin.

7.6.9.4 Location

Influent lines should be located along the bottom of the pond so that the top of the pipe is below the average elevation of the pond seal. However, the pipe shall have adequate seal below it. The use of an exposed dyke to carry the influent line to the discharge points is prohibited.

7.6.9.5 Point of Discharge

The influent line to a square single celled pond should be essentially centre discharging. Each square cell of a multiple celled pond operated in parallel 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 ponds should terminate at approximately the third point farthest from the outlet structure. Influent and effluent piping should be located to minimize short-circuiting within the pond. Consideration should be given to multi-influent discharge points for primary cells of 5 ha or larger.

All aerated cells 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 stabilization basins the inlet pipe may go directly through the dyke and end at the toe of the inner slope.

7.6.9.6 Influent Discharge Apron

Inlet pipes should terminate 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 one metre square shall be provided.

7.6.9.7 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.

7.6.10 Control Structure and Interconnecting Piping

7.6.10.1 Structure

Facility 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.

Control structures shall

(a) be accessible for maintenance and adjustment of controls;

(b) be adequately ventilated for safety and to minimize corrosion;

(c) be locked to discourage vandalism;

(d) contain controls to permit water level and flow rate control, complete shutoff and complete draining;

(e) be constructed of non-corrodible materials (metal-on-metal contact in controls should be of similar alloys to discourage electro-chemical reactions); and

(f) be located to minimize short-circuiting within the cell and avoid freezing and ice damage.

Recommended devices to regulate water level are valves, slide tubes, dual slide gates, or effluent chambers complete with a water level regulating weir. Regulators should be designed so that they can be preset to stop flows at any pond elevation.

7.6.10.2 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.

7.6.10.2.1 Drawdown Structure Piping

a. Submerged Takeoffs

For ponds designed for shallow or variable depth operations, submerged takeoffs are recommended. Intakes shall be located a minimum of three metres from the toe of the dyke and 0.6 metres from the top of the liner and shall employ vertical withdrawal.

b. Multi-level Takeoffs

For ponds that are designed deep enough to permit stratification of pond 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 ponds under warm weather conditions, surface overflow-type withdrawal is recommended. Design should evaluate floating weir box or slide tube entrance with baffles for scum control.

d. Maintenance Drawdown

All ponds shall have a pond drain to allow complete emptying, either by gravity or pumping, for maintenance. These should be incorporated into the above-described structures.

In aerated stabilization basins where a diffused air aeration system and submerged air headers are used, provision should be made to drain each stabilization basin (independently of others) below the level of the air header.

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.

7.6.10.2.2 Hydraulic Capacity

The hydraulic capacity for continuous discharge structures and piping shall allow for at least the expected future peak sewage pumping rate.

The hydraulic capacity for controlled discharge systems shall permit transfer of water at a rate of 150 mm of pond water depth per day at the available head.

7.6.10.2.3 Interconnecting Piping

Interconnecting piping for multiple unit installations operated in series should be valved or provided with other arrangements to regulate flow between structures and permit flexible depth control. The interconnecting pipe to the secondary cell should discharge horizontally near the stabilization basin bottom to minimize need for erosion control measures and should be located as near the dividing dyke as construction permits. Interconnection piping shall enable parallel or series flow patterns between cells.

7.6.10.3 Location

The outlet structure and the inter-connecting pipes should be located i) away from the corners where floating solids accumulate and ii) on the windward side to prevent short-circuiting.

7.6.11 Miscellaneous

7.6.11.1 Fencing

The pond area shall be enclosed with a suitable fence to preclude livestock and discourage trespassing. The fence should be located on the outer dyke at a distance of 500 mm from the top outside edge of the embankment. Fencing should not obstruct vehicle traffic on top of the dyke. A vehicle access gate of sufficient width to accommodate equipment should be provided. All access gates should be provided with locks.

7.6.11.2 Access

An all-weather access road shall be provided to the pond site to allow year-round maintenance of the facility.

7.6.11.3 Warning Signs

Appropriate signs should be provided along the fence around the pond to designate the nature of the facility and warn against trespassing. At least one sign shall be provided on each side of the site and one for every 150 m of its perimeter.

7.6.11.4 Flow Measurement

Provisions for flow measurement shall be provided on the outlet. Safe access to the device should be made to permit safe measurement.

7.6.11.5 Groundwater Monitoring

An approved system of wells or lysimeters may be required around the perimeter of the pond site to facilitate groundwater monitoring. The need for such monitoring will be determined on a case-by-case basis.

7.6.11.6 Pond Level Gauges

Pond level gauges shall be provided.

7.6.11.7 Service Building

A service building for laboratory and maintenance equipment shall be provided, if required.

7.6.11.8 Liquid Depth Operation

Optimum liquid depth is influenced to some extent by stabilization basin area since circulation in larger installations permits greater liquid depth. The basic plan of operation may also influence depth. Facilities to permit operation at selected depths between 0.6 to 1.5 metres 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 metres 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.

7.6.11.9 Pre-Treatment and Post-Treatment

The wastewater shall be treated by bar screens before entering the stabilization basin.

The treated effluent shall be disinfected prior to discharging into the receiving water.

7.7 OTHER BIOLOGICAL SYSTEMS

New biological treatment schemes with promising applicability in wastewater treatment may be considered if the required engineering data for new process evaluation is provided in accordance with Section 4.5.2. 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.

7.7.1 Biological Aerated Filters

Biological aerated filters (BAFs) are submerged, granular media upflow filters, which treat wastewater by biologically converting carbonaceous and nitrogenous matter using biomass fixed to the media and physically capturing 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 countercurrent backwash systems. The co-current backwash design has a nozzle deck supporting a granular media that has a specific gravity greater than 1.0. Pre-treated wastewater is introduced under the nozzle deck and flows up through a slightly expanded media bed, and effluent leaves the filter from above the media. Process air is introduced just above the nozzle deck (the bed is not aerated for denitrification). During backwash, wash water and air scour are introduced below the nozzle deck and flow up through the bed. Wash water is pumped to the head of the plant or directly to solids handling.

The countercurrent backwash BAF operates under the same general principles, except that the granular media has a specific gravity less than 1.0. Therefore, the media float and are retained from above by the nozzle deck. During backwash, wash water flows by gravity through the media. Process air is introduced below the media; therefore, scour air moves countercurrent to the wash water flow.

7.7.1.1 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 2,000 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.

7.7.1.2 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/$m^3 \cdot d$), 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.

7.7.1.3 Performance

The performance of BAFs in terms of allowable loading rates and effluent quality depends on influent wastewater quality and temperature. In general, higher organic or 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:

- warm water temperature,
- adequate aeration and good air distribution, and
- low carbonaceous BOD and suspended solids loading.

Denitrification usually requires methanol addition, and water velocities must be greater than 10 m/h.

7.7.2 Moving Bed Biofilm Reactors

The patented MBBR process was developed by the Norwegian company Kaldnes Miløteknologi (KMT). The basic concept of the MBBR is to have continuously operating, noncloggable 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 plate is placed at the outlet of the reactor. Agitation constantly moves the carrier elements over the surface of the screen; the scrubbing action prevents clogging.

Almost any size or shape tank can be retrofitted with the MBBR process. The filling of carrier elements in the reactor may be decided for each case, based on degree of treatment desired, organic and hydraulic loading, temperature and oxygen transfer capability. The reactor volume is totally mixed and consequently there is no "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.

7.7.3 Membrane Bioreactors

Membrane Bioreactors consist of a suspended growth biological reactor (activated sludge system variation) integrated with a microfiltration membrane system. The key to the technology is the membrane separator which allows elevated levels of biomass to degrade or remove the soluble form of the organic pollutants from the waste stream. These systems typically operate in the nanofiltration or microfiltration range which results in removal of particles greater than 0.01 and 0.1 µm, respectively.

7.7.3.1 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 air flow may be introduced to the bottom of the membrane module to create turbulence which scrubs and cleans the membrane fibres keeping them functioning at a high flux rate. The filtrate or permeate is then collected for reuse or discharge.

Outboard membrane processes operate in a similar manner however, the membranes are contained in a separate tank through which the wastewater requiring filtration constantly flows. Again air is often added for both treatment and membrane scouring purposes. The main difference between the two configurations lies in the membrane cleaning processes where membranes submersed within the aeration tanks must be removed for cleaning while outboard membranes are cleaned by evacuating the membrane tankage and providing for equalization during the cleaning procedures within the main aeration tank.

7.7.3.2 Process Description

The benefits of these processes are consistent effluent quality, reduced footprint, increased expansion capabilities within the same tankage, and ease of operation. Tertiary quality effluent is the normal output of a membrane bioreactor.

Virtually no solids are lost via the permeate stream and the wasting of solids is reduced. As a result, the sludge age can be very accurately determined. Nitrification for ammonia removal is easily achieved by optimizing reactor and sludge age to specific wastewater characteristics and effluent requirements. Absolute control of the nitrifiers results in high nitrification rates even in winter periods and under adverse and unstable conditions.

If required, denitrification can be achieved with ~~for~~ membrane processes because when operating at a MLSS of 15,000 mg/ℓ and higher, the mixed liquor rapidly becomes anoxic in the absence of a continuous stream of air. Furthermore, the high level of biomass ensures that in the anoxic zone, at all times there is enough denitrifiers to efficiently convert the nitrates into nitrogen gas.

7.7.4 Recirculating filters[2]

Recirculating filters provide advanced secondary treatment of settled wastewater or septic tank effluent using sand, gravel or other media. Recirculating filters consist of a lined excavation or structure, filled with uniform washed sand that is placed over an underdrain system. Through a distribution network the wastewater is dosed onto the surface and percolates through the media to the underdrain system. The underdrain system collects filter effluent and directs it to the recirculation tank for further processing or discharge.

7.7.4.1 Recirculating Sand Filters

Recirculating sand filters (RSFs) are aerobic, fixed-film bioreactors. Physical processes that occur in sand filters include straining and sedimentation which remove suspended solids within the pores of the media. Chemical absorption of constituents such as phosphorus also occurs. Bioslimes from the growth of microorganisms develop as films on the sand particle surfaces. As the wastewater percolates through the sand the microorganisms in the slimes absorb the soluble and colloidal waste materials. The absorbed materials are either incorporated into a new cell mass or degraded under aerobic conditions to carbon dioxide and water.

7.7.4.1.1 Applications

Recirculating sand filters can be used for applications including; single-family residences, large commercial establishments, and small communities. They can be used to pretreat wastewater prior to subsurface infiltration and to meet water quality requirements before direct discharge to surface water. RSFs are primarily used to treat domestic wastewater, but they have also been used successfully in treating wastewaters from restaurants and supermarkets, which are high in organic materials. Recirculating filters can be used for both large and small flows and are frequently used where nitrogen removal is necessary.

7.7.4.1.2 System Components

Basic components of recirculating filters include a recirculation/dosing tank, pump and controls, distribution network, filter bed with an underdrain system, and a return line. The return line or the underdrain splits the flow to recycle a portion of the filtrate to the recirculation/dosing tank. A small volume of wastewater and filtrate is dosed to the filter surface on a timed cycle 1 to 3 times per hour. Recirculation ratios are typically between 3:1 and 5:1. The returned aerobic filtrate mixes with the anaerobic septic tank effluent in the recirculation tank before being reapplied to the filter.

There are many types of media used in packed-bed filters. The most common include washed, graded sand however pea gravel has generally replaced it in recent times. Other granular media which can be used include crushed glass, garnet, anthracite, plastic, expanded clay, expanded shale, open-cell foam, extruded polystyrene, and bottom ash from coal-fired power plants. Coarse-fibre synthetic textile materials are also used but are usually restricted to proprietary units.

Recirculation tanks consist of a tank, recirculation pump and controls, and a return filter water flow splitting device. Recirculation tanks store returned filtrate, mix the filtrate with the septic tank effluent, and store peak influent flows. The recirculation pump and controls are designed to dose a constant volume of mixed filtrate and septic tank effluent flow onto the filter on a timed cycle.

Distribution methods used include rigid pipe pressure networks with orifices or spray nozzles, and drip emitters. Rigid pipe pressure networks are the most commonly used method. Orifices with orifice shields, facing upward, minimize hole blockage by stones.

The most common flow splitting devices are ball float valves and proportional splitters. The ball float valve is used where the recirculation tank is designed to remain full. The valve is connected to the return filtrate line inside the recirculation tank. The return line runs through the tank. The ball float valve is open when the water level is below the normally full level. When the tank fills from either the return filtrate or the influent flow, the ball float rises to close the valve, and the remaining filtrate is discharged from the system. The proportional splitters continuously divide the flow between return filtrate and the filtrate effluent. Another type of splitter consists of a sump in which two pipes are stubbed into the bottom with their ends capped. In the crowns of each capped line, a series of equal-sized, pluggable holes are drilled. The return filtrate floods the sump, and the flow is split in proportion to the relative number of holes left open in each perforated capped pipe.

Most RSFs are constructed aboveground and with an open filter surface; however, in cold climates such as in Atlantic Canada, they should be placed in the ground to prevent freezing. The filter basin can be a lined excavation or fabricated tank. Typical liner materials are polyvinyl chloride and polypropylene. The system should be arranged to allow gravity drainage of lines to prevent freezing.

The underdrain system is located on the floor of the tank or lined excavation. The ends of the underdrains should be brought to the surface of the filter and with cleanouts. The underdrain outlet is cut in the basin wall such that the drain invert is at the floor elevation and the filter can be completely drained. The underdrain outlet invert elevation must be sufficiently above the recirculation tank inlet to accommodate a minimum of 0.1 percent slope on the return line and any elevation losses through the flow splitting device. The underdrain is covered with washed, durable gravel to provide a porous medium through which the filtrate can flow to the underdrain system.

Footnote References

1. WEF, "Manual of Practice 8 – Design of Municipal Wastewater Treatment Plants"

2. U.S. Environmental Protection Agency: *Onsite Wastewater treatment Systems Technology Fact Sheet 11, Recirculating Sand/Media Filters.*

8.1 BASIS FOR DISINFECTION OF SEWAGE TREATMENT PLANT EFFLUENT

Disinfection of sewage treatment plant effluent **shall** be required in all cases, unless confirmed otherwise by the regulatory agencies.

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. The designer shall consider the provisions of the Federal "Transportation of Dangerous Goods Act", the Federal "Environmental Protection Act (1999)" (specifically the Environmental Emergency Regulations), the applicable Provincial Dangerous Goods Legislation and the Regulatory Agency having jurisdiction when designing a disinfection system.

8.2 FORMS OF DISINFECTION

Chlorine and ultra violet radiation are the most commonly used methods for wastewater effluent disinfection. The forms most often used for chlorination are liquid chlorine and calcium or sodium hypochlorite. Other disinfectants, including chlorine dioxide, ozone, or bromine~~,~~ may be accepted by the approving authority in individual cases. If chlorination is utilized, it may be necessary to dechlorinate to address concerns related to the negative environmental and health impacts of the release of chlorine into aquatic environments. The use of chlorine capsules along with dechlorination tablets may be considered for small systems.

8.3 CHLORINATION

8.3.1 Design Guidelines

8.3.1.1 *Mixing*

The disinfectant shall be positively mixed as rapidly as possible, with a complete mix being effected in three seconds. This may be accomplished by either the use of a turbulent flow regime or a mechanical flash mixer.

8.3.1.2 *Diffusers*

A chlorine solution diffuser shall be placed ahead of the contact tank and near the vicinity of the mixing area.

8.3.1.3 *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.

A total chlorine residual of 0.5 mg/l is generally required to ensure adequate disinfection of the effluent. To achieve this, 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. Prior to the release of effluent to surface water however, the total chlorine residual should be less than or equal to 0.02 mg/l in order to ensure that elevated concentrations of chlorine are not being discharged. This may necessitate dechlorination as described in Section 8.4.

8.3.1.4 *Coliform Levels*

Acceptable effluent coliform levels shall be based upon the results of the receiving water study and the receiving water quality guidelines or as specified by regulatory agencies having jurisdiction.

8.3.1.5 *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 percent 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.

8.3.2 Chlorination Facilities Design

8.3.2.1 *Feed Equipment*

8.3.2.1.1 *Capacity*

The chlorinator shall be sized according to the design flow of the treatment plant and its capacity may vary, depending on the uses and points of application of the chlorine. For disinfection, the capacity should be adequate to produce a concentration of residual chlorine in the plant effluent as measured by the standard DPD test such as to dependably and consistently obtain 0.5 mg/l chlorine residual after a contact period of 20-30 minutes under average flow conditions.

For normal domestic sewage, the following may be used as a guide in sizing chlorination facilities.

Table 8.1 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

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, or flow proportional.

8.3.2.1.2 Standby Equipment and Spare Parts

Standby equipment of sufficient capacity should be available to replace the largest unit during shutdowns. Spare parts shall be available for all chlorinators to replace parts which are subject to wear and breakage.

8.3.2.1.3 Water Supply

An ample supply of water shall be available for operating the chlorinator. Where a booster pump is required, duplicate equipment should be provided and, when necessary, standby power. Protection of a potable water supply shall conform to the requirements of Section 3.2.9.

The use of a potable water supply for solution-feed chlorinators is the preferred method. However, in the case of small plants, plant effluent may be used for operating the chlorinator.

8.3.2.2 ***Odour Control***

Should odour control be a critical factor, additional capacity of a prechlorination system to the extent of about 80 percent 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 prechlorination and another for disinfection. In the case of large plants, each group shall be interchangeable to facilitate a standby feature.

Prechlorination must be accomplished ahead of the first open structure in the plant and thereby reduce the escape of hydrogen sulphide gas into the atmosphere.

8.3.3 Chlorine Supply

8.3.3.1 *Type*

Chlorine is available for disinfection in gas, liquid (hypochlorite solution), and pellet (Hypochlorite tables) form. The type of chlorine should be carefully evaluated during the facility planning process. The use of chlorine gas or liquid will be most dependent on the size of the facility and the chlorine dose required. Large quantities of chlorine, such as are contained in tonne cylinders and tank cars, can present a considerable hazard to plant personnel and to the surrounding area, should such containers develop leaks. The designer shall consider the provisions of the Federal "Transportation of Dangerous Goods Act", the Federal "Environmental Protection Act (1999)" (specifically the Environmental Emergency Regulations), and the applicable Provincial Dangerous Goods Legislation when designing a disinfection system. Both monetary cost and the potential public exposure to chlorine should be considered when making the final determination.

8.3.3.2 *Cylinders*

Seventy kilogram cylinders should be used when chlorine demand is less than 44 kilograms per day. Cylinders should be stored in an upright position with adequate support brackets and chains at 0.67 cylinder height for each cylinder.

8.3.3.3 *Tonne Containers*

The use of 900 kg containers should be considered where the average daily chlorine consumption is over 44 kilograms. A hoist or crane with a capacity of at least 2000 kg shall be provided for the handling of the tonne containers.

8.3.3.4 *Tank Cars*

At large installations (chlorine consumption greater than 900 kg/day), consideration should be given to the use of multiple containers or tank cars, generally accompanied by liquid chlorine evaporators. Liquid chlorine lines from tank cars to evaporators shall be buried and installed in a conduit and shall not enter below grade spaces. Systems shall be designed for the shortest possible pipe transportation of liquid chlorine.

The tank car being used for the chlorine supply shall be located on a dead end, level track that is a private siding. The tank car shall be protected from accidental bumping by other railway cars by a locked derail device or a closed locked switch or both. The area shall be clearly posted "DANGER-CHLORINE". The tank car shall be secured by adequate fencing with gates provided with locks for personnel and rail access.

The tank car site shall be provided with a suitable operating platform at the unloading point for easy access to the protective housing or the tank car for connection of flexible feedlines and valve operation. Adequate area lighting shall be provided for night time operation and maintenance.

8.3.3.5 *Scales*

Scales of proper size shall be provided at all plants using chlorine gas. At large plants, scales of the indicating and recording type are recommended. At the least a platform scale shall be provided. Scales shall be of corrosion-resistant material. Scales shall be set on grade, or a ramp shall be built to facilitate the moving of cylinders on and off the scale platform. Scales should be provided to accommodate the maximum number of containers on line.

8.3.3.6 ***Evaporators***

Where manifolding of several cylinders or 900 kg containers will be required to supply sufficient chlorine, consideration should be given to the installation of a chlorine evaporator to produce the quantity of gas required.

8.3.3.7 ***Hoists[1]***

Handling of 900 kg containers requires hoisting equipment. It is desirable to use a power-operated hoist and travel particularly when it is necessary to change containers frequently. All hoists must be rated for full load, including the weight of the empty containers and lifting tackle. The cables or chains must not be frayed or damaged.

8.3.3.8 ***Leak Detection and Controls***

A bottle of ammonium hydroxide solution of industrial strength (56%) should be available for detecting chlorine leaks. Where tonne containers or tank cars are used, a leak repair kit approved by the Chlorine Institute shall be provided. Consideration should also be given to the provision of caustic soda solution reaction tanks for absorbing the contents of leaking 900 kg cylinders where such cylinders are in use. At large installations, consideration should be given to the installation of automatic gas detection and related alarm equipment.

8.3.4 **Methods of Dosage Control**

8.3.4.1 ***Open Loop Flow Proportional Control***

Automatic proportioned-to-flow control consists of varying the rate of chlorine feed in proportion to the sewage flow as determined by a metering device. The dosage rate is manually set, and the control device varies the rate in relation to flow rate. The chlorinator may be either automatic or manual start and stop.

Every effort should be made to ensure that the dosing system is as efficient as possible, while maintaining sufficient chlorine residual for disinfection, and to minimize the need for dechlorination of the effluent to achieve the maximum chlorine concentration outlined in Section 8.3.1.3.

8.3.4.2 ***Closed-Loop Flow Proportional Control (Compound-Loop Arrangement with One Chlorine Analyzer):***

Chlorine residual analyzer provides feedback to the chlorinator. Flow signal and dosage signal each separately control the added chlorine feed with a compound-loop arrangement. If the residual is above the pre-determined level, the chlorine feed rate is reduced, and vice versa. In some designs, chlorine residual is measured at one point in the system and in other designs at 2 or 3 points.

8.3.4.3 ***Close-Loop Flow Proportional Control (Compound-Loop Arrangement with Two Chlorine Analyzers***

This ideal system employs quantitative as well and qualitative feed control as in the previous case. However, the qualitative control is accomplished at two points in the flow stream. One sample is automatically collected immediately downstream from the point of chlorination (diffuser) and analyzed by another chlorine analyzer which (i) monitors the combined residual after a given contact time and (ii) adjusts the control point on the analyzer which controls the chlorine metering equipment. When the residual chlorine is more than the desired (preset) level, the chlorine feed rate is reduced and vice versa.

8.3.4.4 *Required Chlorine Control Systems*

Plants with proper qualitative and quantitative control systems are known to chlorinate effectively and efficiently. However, the plants without such controls show either inadequate performance (due to under dosage) or waste chlorine unnecessarily (by undue overdose). Higher than needed chlorine residuals may result in ecological damage to the receiving waters.

The following table summarizes chlorine control guidelines:

TABLE 8.2 - CHLORINE CONTROL GUIDELINES

SIZE OF PLANT	TYPE OF RECEIVING WATER	RECOMMENDED CONTROL	METHOD OF CHLORINE RESIDUAL DETERMINATION
LARGE	ALL TYPES	CLOSED-LOOP, FLOW PROPORTIONAL, TWO CHLORINE ANALYZERS	AMPEROMETRIC TITRATOR; CONTINUOUS DETERMINATION AND RECORDING
MEDIUM	ECOLOGICALLY SENSITIVE WATERS WITH FISHING POTENTIAL	SAME AS LARGE PLANTS	SAME AS LARGE PLANTS
	WATERS OF PUBLIC HEALTH IMPORTANCE	CLOSED-LOOP, FLOW PROPORTIONAL, ONE CHLORINE ANALYZER	AMPEROMETRIC TITRATOR; CONTINUOUS DETERMINATION (OPTIONAL RECORDING)
SMALL	ECOLOGICALLY SENSITIVE RECEIVING WATER	CLOSED-LOOP, FLOW PROPORTIONAL, ONE CHLORINE ANALYZER	AMPEROMETRIC TITRATOR; CONTINUOUS DETERMINATION
	RECEIVING WATER OF PUBLIC HEALTH IMPORTANCE	OPEN-LOOP, FLOW PROPORTIONAL	STARCH-IODIDE METHOD ORRTHOTOLIDINE METHOD (INTERMITTENT - MANUAL)

8.3.5 Storage and Handling

Under the federal “Environmental Emergency Regulations”, anyone storing or using a listed substance above the specified thresholds, or who has a container with a capacity for that substance in excess of the specified quantity, will have to notify Environment Canada of the place where the substance is held, along with the maximum expected quantity and the size of the largest container for that substance. If both the above criteria are exceeded, the regulatee is required to prepare and implement an environmental emergency plan and notify Environment Canada accordingly.

8.3.5.1 *Building Layout and Material Handling*

If gas chlorination equipment and chlorine cylinders are to be in a building used for other purposes, a gas-tight partition shall separate this room from any other portion of the building. Doors to the chlorinator room shall open only to the outside of the building and shall be equipped with panic hardware. Such rooms shall be at ground level and should permit easy access to all equipment.

The storage area should be separated from the feed area. A "DANGER" sign shall be placed on the door and safety precaution instructions to startup and shutdown shall be placed at a visible location on the wall. Full and empty chlorine cylinders shall be stored separately and shall be chained to the wall in the vertical position. Cylinders should not be stored near flammable materials, heating or ventilation units, elevator shafts and on uneven or subsurface floors.

Chlorine cylinders shall be conveyed by a wheeled cart. Suitable lifting bars or hoist shall be used to unload and place the 900 kg containers on their side on level rails.

Storage and handling procedures shall conform to the guidelines of the latest edition of the "Chlorine Manual" prepared by "The Chlorine Institute, Inc." and by the regulatory agency having jurisdiction.

8.3.5.2 ***Special Construction Details***

A clear glass, gas-tight window shall be installed in an exterior door or interior wall of the chlorinator room to permit the chlorinator to be viewed without entering the room. The building shall be of fireproof material. For floor drain information see Section 8.3.7.1

The distance from any point in the room and the outside door shall not exceed five metres. The chlorinator shall be placed 1.0 m from the outside wall. Twenty-five mm piping shall be used and pipes and valves shall be colour coded.

8.3.5.3 ***Heat***

The chlorinator room shall be provided with a means of heating so that a temperature of at least 15℃ can be maintained, but the room should be protected from excess heat. Cylinders shall be kept at essentially room temperature.

8.3.5.4 ***Ventilation***

With chlorination systems, forced, mechanical ventilation shall be installed which will provide one complete air change per minute when the room is occupied. The entrance to the air exhaust duct from the room shall be 300 mm above the floor and the point of discharge shall be so located as not to contaminate the air inlet to any buildings or inhabited areas. The air inlet shall be located near the ceiling on the opposite side of the room so as to provide cross ventilation with air and at such temperatures that will not adversely affect the chlorination equipment. The vent hose from the chlorinator should discharge above grade to the atmosphere. Where public exposure may be extensive such as residential or densely populated areas, scrubbers may be required on ventilation discharge.

8.3.5.5 ***Electrical Controls***

The controls for the fans and lights shall be such that they will automatically operate when the door is open. The controls shall also be such that they can be manually operated from the outside without opening the door. All electrical equipment shall be vapour-proof. Fans and lights should be on the same off and on switch whenever possible.

8.3.5.6 *Respiratory Protection*

A self-contained air-supply breathing apparatus in good operating condition shall be available at all installations where chlorine gas is handled. This equipment shall be stored outside of any room where chlorine is used or stored. Instructions for using, testing and replacing parts and air tanks shall be posted. The units shall use compressed air, have at least 30-minute capacity and be compatible with the units used by the fire department responsible for the plant. Other safety equipment required includes a first-aid kit, a fire extinguisher, goggles and gloves, a chlorine container repair kit and a shower.

8.3.6 Piping and Connections

Piping systems should be as simple as possible, specially selected and manufactured to be suitable for chlorine, with a minimum number of joints; piping should be well supported and protected against temperature extremes.

The correct weight or thickness of steel shall be suitable for use with DRY chlorine liquid or gas. Even minute traces of water added to chlorine results in a corrosive attack that can only be resisted by pressure piping utilizing materials such as silver, gold, platinum or Hasteloy "C". Low pressure lines made of hard rubber, saran-lined, rubber-lined, polyethylene, polyvinylchloride (PVC) or Uscolite materials are satisfactory for wet chlorine or aqueous solutions of chlorine.

Due to the corrosiveness of wet chlorine, all lines designed to handle dry chlorine should be protected from the entrance of water or air containing water.

8.3.7 Miscellaneous

8.3.7.1 *Drains*[2]

Floor drains are not permitted in chlorine gas feed and/or storage rooms, except in installations using evaporators. Where, approved, floor drains must be constructed of corrosion-resistant materials and must discharge to a drainage system separate from the rest of the treatment facility.

8.3.7.2 *Vents*

All chlorinators shall have a pressure vacuum relief vent system, which should be carried to the outside atmosphere, without traps, to a safe area, one vent for each chlorinator. The ends of the vent lines should point down, be covered with a copper wire screen to exclude insects, and should not be more than 7.5 m above the chlorinator. The line should have a slight downward pitch from the high point (directly above the chlorinator) to drain any condensate away from the chlorinator. It is acceptable to run the vent vertically (but no more than 7.5 m) above the chlorinator to the roof, with a 180° return bend at the exit.

Each external chlorine pressure-reducing valve should be checked to see if it is provided with a vent; some are not vented, depending on the chlorine capacity. When supplied, these vents should drain away from the valves. These valves should be located high enough so that the individual drains will have a continuous downgrade to the outside atmosphere.

Evaporators have a steam vapour vent which can be manifolded together and discharged to the atmosphere without traps.

8.3.7.3 *Chlorinator Alarms*

Each chlorinator in large plants shall be equipped with a vacuum switch that should close or open a contact (and start an alarm) when there is an unusually high or low vacuum in the line from the chlorinator to the injector.

Medium size plants are encouraged to include such vacuum switch-alarm systems.

8.3.8 Evaluation of Effectiveness

8.3.8.1 *Sampling*

Facilities shall be included for sampling the disinfected effluent after contact. Either grab or continuous monitoring, as conditions warrant, should be made for effluent chlorine residual of the disinfected effluent.

8.3.8.2 *Testing and Control*

Equipment shall be provided for measuring chlorine residual, employing the standard DPD test. The equipment should enable residual measurement to the nearest 0.1 mg/l in the range below 0.5 mg/l and to an accuracy of approximately 25 percent above 0.5 mg/l. Where the discharge occurs at points requiring rigid bacteriological controls such as on public water supply watersheds, recreational watersheds or shellfish waters or waters tributary thereto, the installation of demonstrated effective facilities for automatic chlorine residual analysis, recording and proportioning systems should be considered. In sensitive areas, dechlorination may be required. Chemicals such as sulphur dioxide, hydrogen peroxide, sodium metabisulphite or granular activated carbon may be used for such purposes.

Where dechlorination is required, equipment should enable a total residual chlorine measurement to 0.02 mg/l. Alternatively, the dechlorination chemical itself can be measured. Any measured amount of dechlorination chemical in the final effluent represents an absence of TRC.

8.3.9 Hypochlorination

8.3.9.1 *Supply*

Hypochlorite for the purpose of wastewater disinfection is usually in a liquid or solid form. Sodium hypochlorite is available in the liquid form with 12 to 15 percent available chlorine. Calcium hypochlorite is a solid with 70 percent available chlorine and comes in either the 22.5 or 45 kg plastic or plastic-lined containers and in pellet form. The choice of chemical is dependent on quantity used and distance of travel from the supplier.

8.3.9.2 *Storage and Handling*

Special storage facilities are required for handling hypochlorination equipment. Chemicals containing chlorine compound should be stored in a separate room used for that purpose only. No other materials should be stored in the same room. The room should be of fire-resistant construction and at or above grade. As heat and light affect the shelf life of sodium hypochlorite, the storage area should be kept cool and be protected from direct sunlight.

Calcium hypochlorite (HTH) shall be kept dry and covered. The storage area must not be serviced by automatic sprinkler systems. When heated above 170℃, HTH releases oxygen. For this reason, HTH must be kept away from flammable materials. Calcium hypochlorite storage areas should be provided with an exhaust system for the purpose of dust removal.

8.3.9.3 *Metering*

Application of hypochlorite for the purpose of disinfection should be by metering pumps specifically designed for this purpose. The system should be capable of maintaining a chlorine residual of at least 0.5 mg/l after a retention time of 20-30 minutes. Calcium hypochlorite should be initially mixed in a make-up tank prior to any chlorination purpose.

8.4 DECHLORINATION

8.4.1 General

Dechlorination of effluent shall be considered when the receiving water is:

a. considered to be highly important for the fishing industry; or

b. ecologically sensitive to chlorine toxicity and susceptible to the adverse effects of chlorine residuals; or

The decisions regarding use of dechlorination shall be made on a case-by-case basis.

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.

8.4.2 Dosage

The dosage of dechlorination chemicals should depend on the residual chlorine in the effluent, the final residual chlorine limit, and the particular form of the dechlorinating chemical used. The most common dechlorinating agent is sulphite. The following forms of the compound are commonly used and yield sulphite (SO_2) when dissolved in water.

Dechlorination Chemical	Theoretical mg/l Required to Neutralize 1 mg/l Cl_2
Sulphur dioxide (gas)	0.9511
Sodium meta bisulphate (solution)	1.34
Sodium bisulphate (solution)	1.4267

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 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.

8.4.3 Containers

Depending on the chemical selected for dechlorination, the storage containers will vary from gas cylinders, liquid in 190 litre 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.

8.4.4 Feed Equipment, Mixing, and Contact Requirements

8.4.4.1 *Equipment*

In general, the same type of feeding equipment used for chlorine gas may be used with minor modifications for sulphur dioxide gas. However, the manufacturer should be contacted for specific equipment recommendations. No equipment should be alternately used for the two gases. The common type of dechlorination feed equipment utilizing sulphur compounds include vacuum solution feed of sulphur dioxide gas and a positive displacement pump for aqueous solutions of sulphite or bisulphate.

The selection of the type of feed equipment utilizing sulphur compounds 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 tonne containers to prevent reliquefaction.

Where necessary to meet the operating ranges, multiple units shall be provided for adequate peak capacity and to provide a sufficiently low feed rate on turn down to avoid depletion of the dissolved oxygen concentrations in the receiving waters.

8.4.4.2 *Mixing Requirements*

The dechlorination reaction with free or combined chlorine will generally occur within 15-20 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.

8.4.4.3 *Contact Time*

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 contract zone. Consideration shall be given to a means of reaeration to assure maintenance of an acceptable dissolved oxygen concentration in the stream following sulfonation.

8.4.4.4 *Standby Equipment and Spare Parts*

The same requirements apply as for chlorination systems.

8.4.4.5 *Sulphonator Water Supply*

The same requirements apply as for chlorination systems.

8.4.5 Housing Requirements

8.4.5.1 *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 4.9. The mixing, storage, and solution delivery areas must be designed to contain or route solution spillage or leakage away from traffic areas to an appropriate containment unit.

8.4.5.2 *Protective and Respiratory Gear*

The respiratory air-pac protection equipment is the same as for chlorine, (See Section 4.9). 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 4.9.

8.4.6 Sampling and Control

8.4.6.1 *Sampling*

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 regulatory agency.

8.4.6.2 *Testing Control*

Provision shall be made for manual or automatic control of sulphonator feed rates based on chlorine residual measurement or flow.

8.4.7 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 flow rate, 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 min with an influent free residual of 3 to 4 mg/l, and an effective carbon bed life of at least 3 years.

8.5 ULTRAVIOLET (UV) DISINFECTION

The following sections describe factors that affect the performance of Ultraviolet Disinfection Systems. Systems should be designed to account for these effects.

8.5.1 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 which absorb or reflect UV light. As the percent transmission gets lower (higher absorbance) the ability of the UV light to penetrate the wastewater and reach target organisms decreases. The system designer must obtain samples of the wastewater during the worst conditions or carefully attempt to calculate the minimum expected UV transmission by testing wastewater from plants which have a similar influent and treatment process. The designer must also strictly define the disinfection limits since they determine the magnitude of the UV dose required.

8.5.2 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.

8.5.3 Design Flow Rate 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. That is, the UV Dose (D) is equal to the intensity (I) times the Retention Time (t).

$$D = It$$

As the flow rate increases the number or size of the UV lamps must be proportionately increased to maintain the same disinfection requirements. An Ultraviolet Disinfection system must be designed for worst case conditions. The minimum dosage occurs at the maximum flow rate and end of lamp life.

8.5.4 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 flow rates. The UV system must be designed for the maximum flow rate. 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 flow rate. 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.

8.5.5 Wastewater Iron Content

Iron can affect the UV disinfection by absorbing UV light. Dissolved iron, iron precipitate on quartz sleeves, and adsorption of iron by suspended solids, bacterial floc and other organic compounds, all decrease UV transmittance.

In cases where the wastewater has an Iron level of > 0.3 mg/L, consideration should be given to pre-treatment or an alternate disinfection system.

8.5.6 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℃, 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.

8.5.7 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 ultraviolet light thereby preventing UV disinfection.

8.5.8 UV Lamp Life

Under certain conditions lamp life for low pressure mercury lamps have been shown to be greater than 13000 hours.[1] 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 percent 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 if 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.

8.5.9 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.

8.6 OZONATION

8.6.1 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 disinfection requirements.

8.6.2 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:

$$\text{Ozone Dosage} = 1.5 + 0.38\ \text{TSS}$$

8.6.3 Design Considerations

8.6.3.1 *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/d 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.

8.6.3.2 *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.

8.6.3.3 *Compressors*

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.

8.6.3.4 *Cooling and Drying*[1]

Pre-treatment for reducing moisture in the feed gas stream shall be required.

Ozone generation is inefficient using current corona discharge technology, with a large portion of the energy consumed being converted to heat. Consequently, cooling of the ozone generator is an important consideration. Cooling systems may include the following:

- A closed circuit using a plate heat exchanger and recirculation pumps,
- A closed circuit with a chiller, or
- A chiller.

Water cooling is typically used, though air-cooled systems are available. The cooling system, regardless of whether it uses air or water, must be closely monitored. This monitoring requires the permanent installation of temperature sensors and recorders at both the influent and effluent ports of the ozonator.

8.6.3.5 *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 are 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.

8.6.3.6 *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.

8.6.3.7 *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 I, may be used in submerged piping, provided the gas temperature is below 60℃ and the gas pressure is low.

8.6.4 Reference Manuals

Environment Canada: Review of Municipal Wastewater Effluent Chlorination/Dechlorination Principles, Technologies and Practices

Water Environment Federation: The Chlorination / Dechlorination Handbook, 2002.

The following sources contain detailed design information for UV Disinfection:

Water Pollution Control Federation: *Wastewater Disinfection*, Manual of Practice FD-10, Alexandria, VA, 1986.

U.S. Environmental Protection Agency: *Design Manual for Municipal Wastewater Disinfection*, EPA 625/ 1-86-021, Cincinnati, OH, 1987.

Footnote References

1. WEF, “Manual of Practice FD – 10, Wastewater Disinfection”, 1996

2. Government of Saskatchewan, Guidelines for Chlorine Gas Use in Water and Wastewater Treatment, 2004.

9.1 PHOSPHORUS REMOVAL

9.1.1 General

9.1.1.1 *Applicability*

The following factors should be considered when determining the need for phosphorus control at wastewater treatment facilities:

a. the present and future phosphorus loadings from the existing municipal wastewater treatment facility to the receiving water;

b. the background phosphorus levels in the receiving water and the effects of these levels on the rate of eutrophication along the entire length of receiving waters;

c. the predicted response of the receiving water to increased phosphorus loadings;

d. the existing and desired water quality of the receiving water along its entire length;

e. the existing and projected uses of the receiving water; and

f. consideration of the best practicable technology available to control phosphorus discharges.

9.1.1.2 *Phosphorus Removal Criteria*

A wastewater treatment facility shall be required to control the discharge of phosphorus if the following conditions exist:

a. 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

b. The wastewater effluent discharge is contributing or may contribute significantly to the rate of receiving water eutrophication.

9.1.1.3 *Method of Removal*

Acceptable methods for phosphorus removal shall include chemical precipitation, high rate filtration or biological processes.

9.1.1.4 *Design Basis*

9.1.1.4.1 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.

9.1.1.4.2 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.

9.1.2 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.

9.1.3 Process Requirements

9.1.3.1 *Dosage*

Typical chemical dosage requirements of various chemicals required for phosphorus removal are outlined in Table 9.1.

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.

9.1.3.2 *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.

TABLE 9.1 - 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
WASTE STABILIZATION PONDS				
SEASONAL RETENTION PONDS	BATCH DOSAGE TO CELLS	ALUM FERRIC CHLORIDE LIME	100-210 17-22 250-350	163 20 300
CONTINUOUS DISCHARGE POND	RAW SEWAGE	ALUM FERRIC CHLORIDE LIME	225 20 400	225 20 400

9.1.3.3 ***Chemical Feed System***

In designing the chemical feed system for phosphorus removal, the following points should be considered:

a. the need to select chemical feed pumps, storage tanks and piping suitable for use with the chosen chemical(s);

b. selection of chemical feed equipment with the required range in capacity;

c. the need for a standby chemical feed pump;

d. provision of flow pacing for chemical pumps proportional to sewage flow rates;

e. flexibility by providing a number of chemical application points;

f. the need for protection of storage and piping from the effect of low temperatures;

g. selection of the proper chemical storage volume;

h. the need for ventilation in chemical handling rooms; and

i. provision for containment of any chemical spills.

9.1.3.4 ***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.

9.1.3.5 ***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.

9.1.3.6 ***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.

9.1.3.7 ***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 Chapter 6. For design of the sludge handling system, special consideration should be given to the type and volume of sludge generated in the phosphorus removal process.

9.1.3.8 ***Filtration***

Effluent filtration shall be considered where effluent phosphorus concentrations of less than 1 mg/ℓ must be achieved.

9.1.4 **Feed Systems**

9.1.4.1 ***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.

9.1.4.2 ***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.

9.1.4.3 *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.

9.1.5 Storage Facilities

9.1.5.1 *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.

9.1.5.2 *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.

9.1.5.3 *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.

9.1.6 Other Requirements

9.1.6.1 *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.

9.1.6.2 *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.

9.1.6.3 *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.

9.1.6.4 *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.

9.1.7 Hazardous Chemical Handling

The requirements of Section 4.9.2 Hazardous Chemical Handling shall be met.

9.1.8 Sludge Handling

9.1.8.1 *General*

Consideration shall be given to the type and additional capacity of the sludge handling facilities needed when chemicals are added.

9.1.8.2 *Dewatering*

Design of dewatering systems should be based, where possible, on an analysis of the characteristics of the sludge to be handled. Consideration should be given to the ease of operation, effect of recycle streams generated, production rate, moisture content, dewaterability, final disposal, and operating cost.

9.2 AMMONIA REMOVAL

9.2.1 Breakpoint Chlorination

9.2.1.1 *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.

9.2.1.2 *Design Considerations*

9.2.1.2.1 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 min is recommended.

9.2.1.2.2 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 pretreatment 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 9.2 shows the quantities of chlorine required, based on operating experience as well as recommended design capabilities. These ratios are applied to the maximum anticipated influent ammonia concentration.

Dechlorination must be considered to minimize the potential for aquatic toxicity due to residual chlorine.

TABLE 9.2 - 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

9.2.1.2.3 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.

9.2.1.2.4 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 prechlorination, 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.

9.2.1.2.5 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.

9.2.2 Air Stripping

9.2.2.1 *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 percent 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.

9.2.2.2 *Design Considerations*

9.2.2.2.1 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.

9.2.2.2.2 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 $l/m^2.s$ removal efficiency is significantly decreased at loadings in excess of 1.3 $l/m^2.s$. 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.

9.2.2.2.3 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.

9.2.2.2.4 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.

9.2.2.2.5 General Construction Features

The stripping tower may be either of the countercurrent (air inlet at base) or cross flow (air inlet along entire depth of fill) type. Generally, provisions should be made to have the capability to recycle tower effluent to increase the removal of ammonia nitrogen during cooler temperatures.

Provisions 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.

9.2.2.2.6 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).

9.3 BIOLOGICAL NUTRIENT REMOVAL

9.3.1 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."

9.3.1.1.1 Mainstream Phosphorus Removal (A/O Process)

The proprietary A/O process is a single sludge suspended-growth system that combines Anaerobic stages and Oxic stages (aerobic) in sequence. Settled sludge is returned to the influent end of the reactor and mixed with the incoming wastewater. Under anaerobic conditions, the phosphorus contained in the wastewater and the recycled cell mass is released as soluble phosphates. Some BOD reduction also occurs in this stage. The phosphorus is then taken up by the cell mass in the aerobic zone. Phosphorus is removed from the liquid stream in the waste activated sludge. The concentration of phosphorus in the effluent is dependent mainly on the ratio of BOD to phosphorus of the wastewater treated.

9.3.1.2 *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.

9.3.1.3 *Design Criteria*

TABLE 9.3 - 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 (d)	2 - 25	10 - 30	
MLSS (mg/ℓ)	2000 - 4000	600 - 5000	2000 - 3000
Hydraulic Retention Time (hrs) Anaerobic Zone Aerobic Zone	 0.5 - 1.5 1 - 3	 8 - 12 4 - 10	 1.8 - 3 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

9.3.2 **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.

9.3.2.1 *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.

9.3.2.1.1 Design Criteria

TABLE 9.4 - DESIGN CRITERIA FOR NITRIFICATION		
DESIGN PARAMETER	SINGLE STAGE	SEPARATE STAGE
Food/Microorganism Ratio (kg BOD_5/kg MLVSS.d)	0.12 - 0.25	0.05 - 0.2
Solids Retention Time (d)	8 - 20	15 - 100
MLSS (mg/ℓ)	1500 - 3500	1500 - 3500
Hydraulic Retention Time (hrs)	6 - 15	3 - 6
Return Activated Sludge (% of Influent Flowrate)	50 - 150	50 - 200

9.3.2.2 *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.

9.3.2.2.1 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.

9.3.2.2.2 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.

9.3.3 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.

9.3.3.1 *A²/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 in the form of nitrate or nitrite is introduced by recycling nitrified mixed liquor from the aerobic section. Effluent phosphorus concentrations of less than 2 mg/l can be expected without effluent filtration; with effluent filtration, effluent phosphorus concentrations may be less than 1.5 mg/l.

9.3.3.2 *Bardenpho Process (5 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.

9.3.3.3 *UCT Process*

The UCT (University of Cape Town) 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.

9.3.3.4 *Design Criteria*

TABLE 9.5 - 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 BOD_5/kg MLVSS.d)	0.15 - 0.25	0.1 - 0.2	0.1 - 0.2	0.1
Solids Retention Time (d)	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	--

9.3.4 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 or nitrogen removal can also be accomplished. Overall cycle time may vary from 3 to 24 hours. A carbon source in the anoxic phase is required to support denitrification-either an external source or endogenous respiration of the existing biomass.

9.3.5 Detailed Design Manuals

The following sources contain detailed design information for biological nutrient removal:

Water Pollution Control Federation: *Nutrient Control, Manual of Practice* FD-7, Washington, DC, 1983.

U.S. Environmental Protection Agency: *Design Manual for Phosphorus Removal*, EPA 625/ 1-87-001, Cincinnati, OH, 1987.

U.S. Environmental Protection Agency: *Process Design Manual for Nitrogen Control*, Office of Technology Transfer, Washington, DC, October 1975.

U.S. Environmental Protection Agency: *Process Design Manual for Phosphorus Removal*, Office of Technology Transfer, Washington, DC, April 1976.

Environment Canada: *Treatment Processes for the Removal of Ammonia from Municipal Wastewater,* 2003

9.4 EFFLUENT FILTRATION

9.4.1 General

9.4.1.1 *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 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.

9.4.1.2 *Design Considerations*

Factors to consider when choosing between the different filtration systems which are available, include the following:

a. the installed capital and expected operating and maintenance costs;

b. the energy requirements of the systems (head requirements);

c. the media types and sizes and expected solids capacities and treatment efficiencies of the system; and

d. 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.

9.4.2 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 with dechlorination used as required to ensure protection of aquatic life.)

9.4.3 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.

9.4.4 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.

9.4.5 Filtration Rates

9.4.5.1 *Hydraulic Loading Rate*

Filtration rates at peak hourly sewage flow rates, including backwash flows, should not exceed 2.1 $l/m^2 \cdot s$ 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 $l/m^2 \cdot s$ 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.

9.4.5.2 *Organic Loading Rate*

Peak solids loading rate should not exceed 50 $mg/m^2 \cdot s$ for shallow bed filters and 80 $mg/m^2.s$ 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).

9.4.6 Backwash

9.4.6.1 *Backwash Rate*

The backwash rate shall be adequate to fluidize and expand each media layer a minimum of 20 percent based on the media selected. The backwash system shall be capable of providing a variable backwash rate so that the maximum rate is at least 14 $l/m^2 \cdot s$ and a minimum backwash period of 10 minutes.

9.4.6.2 *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 percent 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.

9.4.7 Filter Media

9.4.7.1 *Selection*

Selection of proper media size will depend on the filtration rate selected, the type of treatment provided prior to filtration, filter configuration, and effluent quality objectives. In dual or multi-media filters, media size selection must consider compatibility among media.

9.4.7.2 *Media Specifications*

The following table 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 9.6 MEDIA DEPTHS AND SIZES

(Minimum Depth)
(Effective Size)

	Single Media	Multi-Media (2 media)	Multi-Media (3 media)
Anthracite	-	50 cm 1.0 - 2.0 mm	50 cm 1.0 - 2.0 mm
Sand	120 cm 1.0 - 4.0 mm	30 cm 0.5 - 1.0 mm	25 cm 0.6 - 0.8 mm
Garnet or Similar Material	-	-	5 cm 0.3 - 0.6 mm
Uniformity Coefficient shall be 1.7 or less			

9.4.8 Filter Appurtenances

The filters shall be equipped with washwater 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.

9.4.9 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.

9.4.10 Backwash Surge Control

The rate of return of waste filter backwash water to treatment units should be controlled such that the rate does not exceed 15 percent 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.

9.4.11 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.

9.4.12 Proprietary Equipment

Where proprietary filtration equipment not conforming to the preceding requirements is proposed, data which supports the capability of the equipment to meet effluent requirements under design conditions shall be provided. Such equipment will be reviewed on a case-by case basis at the discretion of the regulatory agencies.

9.5 MICROSCREENING

9.5.1 General

9.5.1.1 *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.

9.5.1.2 *Design Considerations*

Pilot plant testing on existing secondary effluent is encouraged. Where pilot studies so indicate, where microscreens follow trickling filters or ponds, or where effluent 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.

9.5.2 Screen Material

The microfabric 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.

9.5.3 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 $l/m^2{\cdot}s$ 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.

9.5.4 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 $l/m{\cdot}s$ 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 percent 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.

9.5.5 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.

9.5.6 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.

9.6 ACTIVATED CARBON ADSORPTION

9.6.1 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.

9.6.2 Design Considerations

The usefulness and efficiency of carbon adsorption for wastewater treatment depends on the quality and quantity of the delivered wastewater. To be fully effective, the carbon unit should receive an effluent of uniform quality, without surges in the flow. Other wastewater qualities of concern include 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:

a. attraction of carbon for solute;
b. attraction of carbon for solvent;
c. solubilizing power of solvent or solute;
d. association;
e. ionization;
f. effect of solvent on orientation at interface;
g. competition for interface in presence of multiple solutes;
h. coadsorption;
i. molecular size of molecules in the system;
j. pore size distribution in carbon;
k. surface area of carbon; and
l. 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.

9.6.3 Unit Sizing

9.6.3.1 *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 min 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 min should be used where the desired effluent quality is a COD of 10 to 20 mg/l, and 30 to 35 min when the desired effluent COD is 5 to 15 mg/l. For chemical-physical treatment plants, carbon contact times of 20 to 35 min should be used, with a contact time of 30 min being typical.

9.6.3.2 *Hydraulic Loading Rate*

Hydraulic loading rates of 2.5 to 7.0 $l/m^2 \cdot s$ 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 $l/m^2 \cdot s$ are used. Actual operating pressure seldom rises above 7 kN/m^2, for each 0.3 m of bed depth.

9.6.3.3 *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 percent 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.

9.6.3.4 *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.

9.6.4 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 min.

The normal quantity of backwash water employed is less than 5 percent of the product water for a 0.8 m deep filter and 10 to 20 percent 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 $l/m^2 \cdot s$.

9.6.5 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.

9.6.6 Instrumentation

The individual carbon columns should be equipped with flow and headloss measuring devices.

9.6.7 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.

9.6.8 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 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.

9.6.9 Carbon Regeneration

9.6.9.1 *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 9.7.

TABLE 9.7 – TYPICAL CARBON DOSAGES FOR DIFFERENT COLUMN WASTEWATER INFLUENTS

PRETREATMENT	TYPICAL CARBON DOSAGE REQUIRED PER m^3 OF COLUMN THROUGHPUT (g/m^3)*
COAGULATED, SETTLED AND FILTERED ACTIVATED SLUDGE EFFLUENT	35 - 70
FILTERED SECONDARY EFFLUENT	70 - 100
COAGULATED, SETTLED, AND FILTERED RAW WASTEWATER (PHYSICAL - CHEMICAL)	100 - 300

**LOSS OF CARBON DURING EACH REGENERATION CYCLE TYPICALLY WILL BE 5 TO 10 PER CENT. MAKE-UP CARBON IS BASED ON CARBON DOSAGE AND THE QUALITY OF THE REGENERATED CARBON*

9.6.9.2 *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.

9.6.9.3 *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 percent 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.

9.7 CONSTRUCTED WETLANDS

9.7.1 General

Constructed wetlands are inundated land areas with water depths typically less than 0.6 m that support the growth of emergent plants such as cattail, bulrush, reeds, and sedges. The vegetation provides surface for the attachment of bacterial films, aids in the filtration and adsorption of wastewater constituents, transfers oxygen into the water column, and controls the growth of algae by restricting the penetration of sunlight.

Although plant uptake is an important consideration in contaminant removal, particularly nutrient removal, it is only one of many active removal mechanisms in the wetland environment. Removal mechanisms have been classified as physical, chemical and biological and are operative in the water column, the humus and soil column beneath the growing plants, and at the interface between the water and soil columns. 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.

9.7.2 Types

Wastewater treatment systems using constructed wetlands have been categorized as either free water surface (FWS) or subsurface flow (SF) types.

a. 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.

b. Subsurface Flow Wetlands (SF)

A SF wetland system consists of channels or basins that contain gravel or sand media which will support the growth of emergent vegetation. The bed of impermeable material is sloped typically between 0 and 2 percent. 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.

9.7.3 Site Evaluation

Site characteristics that must be considered in wetland system design include topography, soil characteristics, existing land use, flood hazard, and climate.

a. 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 (SF) are normally designed and constructed with slopes of 1 percent 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 percent.

b. Soil

Sites with slowly permeable ($< 1.4 \times 10^{-4}$ cm/sec) surface soils or subsurface layers are most desirable for wetland systems because the objective is to treat the wastewater in the water layer above the soil profile. Therefore, percolation losses through the soil profile should be minimized. As with overland-flow systems, the surface soil will tend to seal with time due to deposition of solids and growth of bacterial slimes. Permeabilities of native soils may be purposely reduced by compacting during construction. Sites with high permeability soils may be used for small systems by constructing basins with clay or artificial liners. The depth of soil to groundwater should be a minimum of 0.3 - 0.6 m to allow sufficient distance for treatment of any percolate entering the groundwater.

c. Flood Hazard

Wetland sites should be located outside of flood plains, or protection from flooding should be provided.

d. 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.

e. 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.

9.7.4 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.

9.7.5 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 SF 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 SF systems because the depth of rhizome penetration allows for the use of deeper basins.

Harvesting of wetland vegetation is generally not required, especially for SF systems. However dry grasses in FWS systems are burned off periodically to maintain free-flow conditions and to prevent channeling of the flow. Removal of the plant biomass for the purpose of nutrient removal is normally not practical.

9.7.6 Design Parameters

9.7.6.1 *Detention Time*

a. Free Water Surface Wetlands (FWS)

The relationship between BOD removal and detention times for FWS is represented by the equation:

C_e = Co exp ($-k_T$ t)]

where :

C_e = effluent BOD, mg/l
C_o = influent BOD, mg/l

k_T = temperature dependent rate constant, d^{-1}
= k_{20} x $1.06^{(T-20)}$
k_{20} = d^{-1}
T = average monthly water temperature, ℃

t = average detention time, d
= $A_S cy/Q_A$

A_S = design surface area of wetland, m^2
c = fraction of cross sectional area not used by plants
y = depth of water in the wetland, m
Q_A = average flow through the wetland [(Qin + Qout) / 2], m^3/d

b. Subsurface Flow Wetlands (SF)

The relationship between BOD removal and detention times for SF is represented by the equation:

C_e = Co exp ($-K_T t'$)

where

C_e = effluent BOD, mg/l
C_o = influent BOD, mg/l

K_T = temperature dependent rate constant, d^{-1}
= K_{20} x $1.06^{(T-20)}$
T = average monthly water temperature, ^oC
K_{20} = d^{-1}

t'	=	$A_s \alpha y / Q_A$
A_S	=	design surface area of wetland, m^2
α	=	porosity of basin medium (See Table 9.8 for media characteristics)
y	=	depth of water in the wetland, m
Q_A	=	average flow through the wetland [(Qin + Qout) / 2], m^3/d

Note: See Table 9.9 for typical parameters for FWS and SF wetlands

9.7.6.2 *Water Depth*

For FWS, the design water depth depends on the optimum depth for the selected vegetation. In cold climates, the operating depth is normally increased in the winter to allow for ice formation on the surface and to provide the increased detention time required at colder temperatures. Systems should be designed with an outlet structure that allows for varied operating depths.

The design depth of SF systems is controlled by the depth of penetration of the plant rhizomes and roots because the plants supply oxygen to the water through the root/rhizome system.

See Table 9.9 for typical FWS and SF water depths.

9.7.6.3 *Hydraulics and Hydrological Considerations*[1]

Manning's equation is generally accepted as a model for the flow of water through FWS wetland systems. Flow velocity depends on depth of the water, hydraulic gradient (i.e., slope of the water surface), and the resistance to flow.

$$v = (1/n)(y^{2/3})(s^{1/2})$$

Where
v = flow velocity, m/s
n = Manning's coefficient, $s/m^{1/3}$
s = hydraulic gradient, m/m
y = water depth, m

The relationship between Manning's n coefficient and the resistance factor is defined as

$$n = a/y^{1/2}$$

Where
a = is the resistance factor, $s \cdot m^{1/6}$

Reed et al. (1995) presented the following values for a in FWS wetlands.
Sparse, low standing vegetation, y > 0.4 m:
$a = 0.4\ s \cdot m^{1/6}$
Moderately dense vegetation, y ≥ 0.3 m:
$a = 1.6\ s \cdot m^{1/6}$
Very dense and litter, y < 0.3 m
$a = 6.4\ s \cdot m^{1/6}$

The aspect ratio (i.e. length: width ratio) selected for a FWS wetland can influence the hydraulic regime because resistance to flow increases as length increases. Reed et al. (1995) developed a model that can estimate the maximum desirable length of an FWS wetland channel.

$$L = [(A_S)(y^{2.667})(m^{0.5})(86400)/(a)(Q_A)]^{0.667}$$

Where:

L = maximum length of wetland cell, m;

A_S = design surface area of wetland, m^2;

y = depth of water in the wetland, m;

m = portion of available hydraulic gradient used to provide the necessary head, percent as a decimal;

a = resistance factor, $s \cdot m^{1/6}$

Q_A = average flow through the wetland, m^3/d

$= (Q_{IN} + Q_{OUT})/2$

An initial m value between 10 and 20% is suggested for design to ensure a future reserve as a safety factor. In the general case this model produces an aspect ratio of 3:1 or less. Using the average flow Q_A in the model compensates for the influence of precipitation, evapotranspiration, and seepage on the flow through the wetland. The design surface area A_s is the bottom area of the wetland.

Darcy's Law describes the flow regime in a porous media and is generally accepted for the hydraulic design of SF wetlands.

Because $\upsilon = -k_S s = Q_A/A_c y$

Therefore $Q_A = k_s A_c s$

Where

Q_A = average flow through the SF wetland, m^3/d

K_S = hydraulic conductivity of a unit area of the wetland perpendicular to the flow direction, $m^3/m^2/d$

A_c = total cross-sectional area perpendicular to flow, m^2

s = hydraulic gradient or slope of the water surface in the wetland, m/m

υ = Darcy's velocity, the apparent flow velocity through the-cross sectional area.

The aspect ratio (i.e. length: width ratio) selected for a SF wetland can influence the hydraulic regime because resistance to flow increases as length increases. Reed et al. (1995) developed a model that can estimate the maximum desirable length of an SF wetland channel.

$$W = (1/y)[(Q_A)(A_s)/(m)(k_s)]^{0.5}$$

Where:

W = maximum width of the SF wetland cell, m;

A_S = design surface area of wetland, m^2;

y = depth of water in the wetland, m;

m = portion of available hydraulic gradient used to provide the necessary head, percent as a decimal;

k_S = hydraulic conductivity of the media used, $m^3/m^2/d$
Q_A = average flow through the wetland, m^3/d
= $(Q_{IN} + Q_{OUT})/2$

The *m* value in the equation above ranges from 5 to 20% of the potential head available. For large projects, the hydraulic conductivity k_s should be directly measured with a sample of the media to be used. When using the maximum width equation, not more than one-third of the effective hydraulic conductivity k_s should be used in the calculation, and *m* value should not exceed 20% to ensure a large safety factor against potential clogging and other contingencies not defined at the time of design. Table 9.8 gives the typical characteristics for media in SF wetlands.

TABLE 9.8 – TYPICAL MEDIA CHARACTERISTICS FOR SF WETLANDS

Media type	D_{10} Effective size, mm	Porosity, α	K_s, $m^3/m^2 \cdot d$
Course Sand	2	0.28-0.32	100-1000
Gravelly sand	8	0.30 - 0.35	500-5000
Fine gravel	16	0.35 - 0.38	1000-10,000
Medium gravel	32	0.36 - 0.40	10,000-50,000
Coarse rock	128	0.38 – 0.45	50,000-250,000

TABLE 9.9 – TYPICAL PARAMETERS FOR FWS AND SF WETLANDS

Parameter	FWS Wetland	SF
Porosity (α)	0.65 to 0.75	0.35 to 0.45
Depth (y), m	0.15 to 0.60	0.30 to 0.60
Fraction of cross sectional area not used by plants (c)	0.65 to 0.75	0.65 to 0.75
BOD_5 Removal		
K_{20}, d^{-1}	0.678	1.104
θ	1.06	1.06
Background Concentration, mg/l	6	6
TSS Removal		
C_e/C_o	[0.1139 + 0.00213(HLR)]	[0.1058 + 0.0011(HLR)]
HLR = hydraulic loading rate, mm/d x 0.1	-	-
TSS removal does not depend on temperature		
Background Concentration, mg/l	6	6
Ammonia Removal		
At 0℃, $K_T(d^{-1})$	0	0
At 1℃, K_{20}	0.2187	$(K_{NH})(\theta)^{(T-20)}$
θ	1.048	1.048
K_{NH} = rate constant 20℃ for SF wetlands, d^{-1}(*rz* = portion of SF bed occupied by plant roots, %	-	$K_{NH} = 0.1854 + 0.3922(rz)^{(2.6077)}$

TABLE 9.9 – TYPICAL PARAMETERS FOR FWS AND SF WETLANDS

Parameter	**FWS Wetland**	**SF**
as a decimal can equal 0 to 0.1, depending on root depth (0.5 is typical))		
Background Concentration	0.2	0.2
Note: It is prudent to assume that all TKN entering the wetland can appear as ammonia; so assume C_o for ammonia is equal to influent TKN.		
Nitrate Removal		
At 0℃, $K_T(d^{-1})$	0	0
At 1℃+, K_{20}	1.0	1.0
θ	1.15	1.15
Background Concentration, mg/l	0.2	0.2
Note: It is conservative to assume that all ammonia removed in the previous step can appear as nitrate; so C_o for nitrate removal design equals C_e from ammonia removal plus nitrate present in the influent.		
TN Removal		
Effluent $T_N = C_{e(NO3)} + (C_{e(NH4)} – C_{e(NO3)})$		
Background Concentration, mg/l	0.4	0.4
Note: A specific model for total nitrogen removal is not available in this set. The effluent TN can be estimated as the sum of residual ammonia and remaining nitrate (C_o - C_e)		
Total Phosphorus Removal		
	$C_e/C_o = \exp(-K_p/HLR)$	$C_e/C_o = \exp(-K_p/HLR)$
K_p, mm/d x 0.1	2.73	2.73
TP removal does not depend on temperature		
HLR = average hydraulic loading rate, cm/d		
Background Concentration, mg/l	0.5	0.5
Fecal Coliform Removal		
C_e/C_o mpn/100mL	$[1/1 + K_T(t/x)]^x$	$[1/1 + K_T(t/x)]^x$
K_{20}, d^{-1}	2.6	2.6
θ	1.19	1.19
t,d	HRT in the system	HRT in the system
x	numbers of wetland cells in series	numbers of wetland cells in series
Background Concentration, cfu/100 mL	2000	2000
Note: This model was developed for facultative ponds and is believed to give a conservative estimate for fecal coliform removal in both FWS and SF wetlands.		

9.7.7 Vector Control

FWS systems provide ideal breeding habitat for mosquitoes. Plans for biological control of mosquitoes through the use of fish that prey on mosquito larvae, ~~fish~~ and swallows should be incorporated in the design. Thinning of vegetation may also be necessary to eliminate pockets of water that are inaccessible to fish.

Mosquito breeding should not be a problem in SF systems, provided the system is designed to prevent mosquito access to the subsurface water zone. The surface is normally covered with pea gravel or coarse sand to achieve this purpose.

9.7.8 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.

9.7.9 Monitoring

Monitoring is necessary to maintain loadings within design limits. A routine monitoring program should be established for the following parameters:

a. wastewater application rates ($m^3/m^2{\bullet}d$);

b. discharge flow rates (m^3/d);

c. wastewater quality, including BOD_5 and COD, suspended solids, total dissolved solids, total nitrogen, total phosphorous, pH and sodium adsorption ratio; and

d. discharge water quality according to the analyses summarized in item (c).

9.8 FLOATING AQUATIC PLANT TREATMENT SYSTEMS

9.8.1 General

Aquatic treatment systems consist of one or more shallow ponds in which one or more species of water tolerant vascular plants such as duckweed is 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.

9.8.2 Plant Selection

The principal floating aquatic plants used in aquatic treatment systems are, duckweed and pennywort. These plants are described in greater detail in the following discussion.

9.8.2.1 Duckweed

Duckweed are small, green freshwater plants with fronds from one to a few millimetres in width with a short root, usually less than 12 mm in length. Duckweed are the smallest and the simplest of the flowering plants and have one of the fastest reproduction rates. Duckweed grown in wastewater stabilization pond effluent (at 27℃) doubles in frond numbers, and therefore in area covered, every four days. The plant is essentially all 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.

9.8.2.2 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. 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 $kg/m^2{\cdot}d$ in warm climates. 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 lettuce biomass production systems.

9.8.3 Types of Systems

The principal types of floating aquatic plant treatment systems used for wastewater treatment are those employing duckweed.

9.8.3.1 Duckweed Systems

Duckweed and pennywort have been used primarily to improve the effluent quality from facultative ponds or stabilization ponds by reducing the algae concentration. Conventional pond 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.

9.8.4 Climatic Constraints

Duckweed is cold tolerant and can be grown practically at temperatures as low as 7℃.

9.8.5 Preapplication Treatment

The minimum level of preapplication 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 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 preapplication treatment step because phosphorus removal in aquatic treatment systems is minimal.

9.8.6 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 duckweed systems are summarized in Table 9.10 for different levels of pre-application treatment.

TABLE 9.10- FLOATING AQUATIC PLANT SYSTEM DESIGN CRITERIA

ITEM	DUCKWEED TREATMENT SYSTEM
Influent Wastewater	Facultative Pond Effluent
Influent BOD_5 (mg/l)	40
BOD_5 Loading (kg/ ha·d)	22 - 28
Water Depth (m)	1.3 - 2.0
Detention Time (d)	20 - 25
Hydraulic Loading Rate (m^3/ ha·d)	570 - 860
Water Temperature (℃)	>7
Harvest Schedule	Monthly

9.8.7 Pond Configuration

9.8.7.1 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.

9.8.8 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. Significant phosphorus removal is achieved only with frequent harvesting. Duckweed harvesting for nutrient removal may be required as often as once per week during warm periods.

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.

9.8.9 Detailed Design Guidelines

The following sources contain detailed design information for natural wastewater treatment systems:

Water Pollution Control Federation: *Natural Systems for Wastewater Treatment*, Manual of Practice FD-16, Alexandria, VA, 1990.

U.S. Environmental Protection Agency: *Design Manual for Constructed Wetlands and Aquatic Plant Systems for Municipal Wastewater Treatment*, EPA 625/ 1-88-022, Cincinnati, OH, 1988.

Footnote References

1. Water Environment Federation: *Natural Systems for Wastewater Treatment*, Manual of Practice FD-16, Alexandria, VA, 2001.

10.1 GENERAL

With many communities throughout the world approaching or reaching the limits of their available water supplies, reclaimed water use has become an attractive option for conserving and extending available water supplies. Reclaimed water use is the controlled application of treated wastewater by irrigation (onto the land surface to achieve disposal, utilization, and/or treatment of the wastewater) or by infiltration (into the soil). This can be achieved by a number of options including land application, surface and subsurface irrigation, in-ground trenches, and overland flow, as approved by regulatory agencies having jurisdiction.

Water reclamation and nonpotable reuse only require conventional water and wastewater treatment technology that is widely practised and readily available in countries throughout the world. Furthermore, because properly implemented nonpotable reuse does not entail significant health risks, it has generally been accepted and endorsed by the public in the urban and agricultural areas where it has been introduced. This section provides information on planning considerations re-use applications, water quality considerations, and guidelines for wastewater irrigation and other re-use criteria.

In addition to the general pre-design report requirements, the designer shall include supplemental information as outlined in Section 1.3.6.

10.1.1 DEFINITIONS[1]

Biological Oxygen Demand (BOD)
A measure of the quantity of oxygen used in the biochemical oxidation of organic matter in a specified time, at a specific temperature, and under specified conditions.

Carbonaceous Biochemical Oxygen Demand (CBOD)
A quantitative measure of the amount of dissolved oxygen required for biological oxidation of carbon-containing compounds in a sample.

Chemical Oxygen Demand (COD)
A quantitative measure of the amount of oxygen required for the chemical oxidation of carbonaceous (organic) material in wastewater using inorganic dichromate or permanganate salts as oxidants in a 2-hour test.

Infiltration

The flow or movement of water through interstices ot pores pf soil or other porous medium.

Irrigation

The artificial application of water to lands to meet the water needs of growing plants not met by rain fall.

10.2 TREATED EFFLUENT APPLICATION METHODS

10.2.1 General

Land application of treated sewage effluent is a method of disposing of effluent without direct discharge to surface waters. Ground disposal installations are normally used where the waste contains pollutants which can successfully be removed through distribution to the soil mantle. These pollutants can be removed through organic decomposition in the vegetation-soil complex and by adsorptive, physical, and chemical reactions with earth materials. Preliminary considerations of a site for ground disposal should include the compatibility of the waste with the organic and earth materials and the percolation rates and exchange capacity of the soils. The ground disposal of treated effluent will eventually recharge the local groundwater; therefore, the quality, direction and rate of movement, and local use of the groundwater, present and potential, are prime considerations in evaluating a proposed site.

It is essential to provide good vegetation growth conditions and removal of nutrients. It must be realized that a groundwater mound will develop below the application area after it is in use. The major factors in design of ground disposal fields are topography, soils, geology, hydrology, weather, agricultural practice, adjacent land use, and equipment selection and installation.

The primary methods used for distributing treated effluent on the land are irrigation, and infiltration.

Table 10.1 outlines various features and performance of treated effluent land application systems.

TABLE 10.1 – COMPARISON OF FEATURES AND PERFORMANCE FOR TREATED EFFLUENT UTILIZATION, TREATMENT AND DISPOSAL SYSTEMS		
SYSTEM REQUIREMENT	STANDARD RATE IRRIGATION	RAPID INFILTRATION
SOIL PERMEABILITY	MODERATE (MEDIUM TEXTURE SOIL)	RAPID (LOAMY SANDS AND GRAVELS)
UTILIZATION OF WATER AND NUTRIENTS	HIGH	NONE
SLOPE	UP TO 30% FOR SPRINKLER AND 6% FOR SURFACE METHODS	NOT CRITICAL
STORAGE	HIGH (7-9 MONTHS)	NIL
LAND AREA	HIGH	LOW
WATER QUALITY – SALINITY, ETC.	VERY HIGH	MEDIUM TO LOW
TREATMENT EFFICIENCY	VERY HIGH	MEDIUM
LOADING RATE	500 – 6000 $l/m^2 \cdot a$	6000 – 100,000 $l/m^2 \cdot a$

10.2.2 Irrigation

10.2.2.1 Piping to Sprinklers

The piping should be arranged to allow the irrigation pattern to be varied easily. Stationary systems are preferred; but if a moveable system is proposed, one main header must be provided with individual connections for each field and sufficient spare equipment must be available to assure non-interrupted irrigation. Facilities must be provided to allow the pipes to be completely drained at suitable points to prevent freezing and spillage of treated effluent into sensitive areas.

10.2.2.2 Sprinkling System

Sprinklers should be located to give a non-irrigated buffer zone around the irrigated area, and design of the buffer zone should consider wind transport of the treated effluent. The system shall be designed to provide an even distribution over the entire field.

The selected application rate should be low enough to allow the irrigated treated effluent to percolate into the soil and to assure proper residency within the soil mantle. Proposed application rates will not be accepted without substantiating data.

In general, sufficient monitoring controls should be provided to indicate the degree of efficiency with which the sprinklers are working. A pressure gauge and flow meter should be provided.

10.2.2.3 Site Buffer Zone

The requirements for buffer zones around the irrigation operation are outlined in Section 10.3.3.2, and are dependent on a number of site specific factors.

10.2.3 Rapid Infiltration (RI)

10.2.3.1 Applicability

Rapid infiltration (RI) involves the application of treated effluent to land by means of basins. The treated effluent percolates through the soil, undergoes a variety of physical, chemical and biological reactions and eventually reaches the groundwater. The loss of water via plants or evaporation is minor compared to the loss by percolation. The loading must be intermittent to allow for the restoration of aerobic conditions in the soil. Acceptable salinity, boron, nitrogen and phosphorus levels in the treated effluent will be governed by the potential use of the groundwater downstream of the RI site. The permeability of the site is, however, very important to the performance of a RI system. Therefore, the sodium adsorption ratio of the effluent should be below 9.

Optimum site conditions for rapid infiltration (RI) are dependent upon the quantity of wastewater to be treated and the degree of treatment required. Generally, there will be an inverse relationship between maximum wastewater application rate and the degree of treatment. Soil conditions required for a good RI site are a deep uniform sandy loam to loamy sand having the following chemical characteristics;

pH	6.0 - 8.5
Organic Matter	0.5 - 3.0%
Electrical Conductivity	2 dS/m
Sodium Adsorption Ratio	10
Cation Exchange Capacity	10 meq/100 g

Free Ca or Mg CO_3 should be present

Rapid infiltration installations require permeable granular subsurface materials. A minimum of 4 m separation between the water table and the basin bottom between irrigation cycles is recommended. In situations where potable water systems will not be affected and tertiary treatment is provided, the 4 m vertical separation distance may be reduced. As a minimum, the separation distance should be 1 m between the water table and the bottom of the basin during operation. Adverse natural groundwater conditions can be modified by the installation of underdrains and/or recovery wells.

Excessive slopes will restrict the usefulness of a RI site. The maximum slope is that which maintains downward infiltration with no premature lateral discharge. Generally, the maximum slope is 5% unless considerable earth moving is undertaken. Uniform flat topography will reduce construction costs. In areas where facultative lagoons are used for treatment, the lagoons will generally be large enough to provide cold weather storage. However, the infiltration area will have to be large enough to treat the annual wastewater production during the warm weather period. Treated effluent from treatment plants with short detention times will retain sufficient heat to allow continuous RI treatment and eliminate the need for storage.

10.2.3.2 *Area and Infiltration Rate*

Prior to site selection the planner must determine the approximate land area required for an RI system. This can be obtained by using sewage flow data and the annual amount of infiltration per unit area. The hydraulic conductivity required to estimate total infiltration can be determined from Table 10.2 and the following calculations. It is then suggested that a factor of 1.5 be applied to the calculated area requirements.

TABLE 10.2 – HYDRAULIC CONDUCTIVITIES OF VARIOUS GRANULAR DEPOSITS

Deposit	Hydraulic Conductivity (cm/s)
Clean, well sorted sand and gravel	10^{-1}
Clean sand, moderately sorted gravel	10^{-2}
Moderately sorted sand and gravel	10^{-3}
Poorly sorted sand and gravel	10^{-4}

Infiltration capacity is estimated by the following procedure:

a. Estimate site hydraulic conductivity, in cm/s.

b. Determine annual hydraulic loading and convert ℓ/m²•d to m/a (multiply by 0.365).

c. Interpolate the site area (on the y-coordinate of Figure 10.1) using the line most closely representing the estimated hydraulic loading rate determined. The site area can also be determined by dividing the annual average treated effluent flow rate by the design annual hydraulic loading as given below:

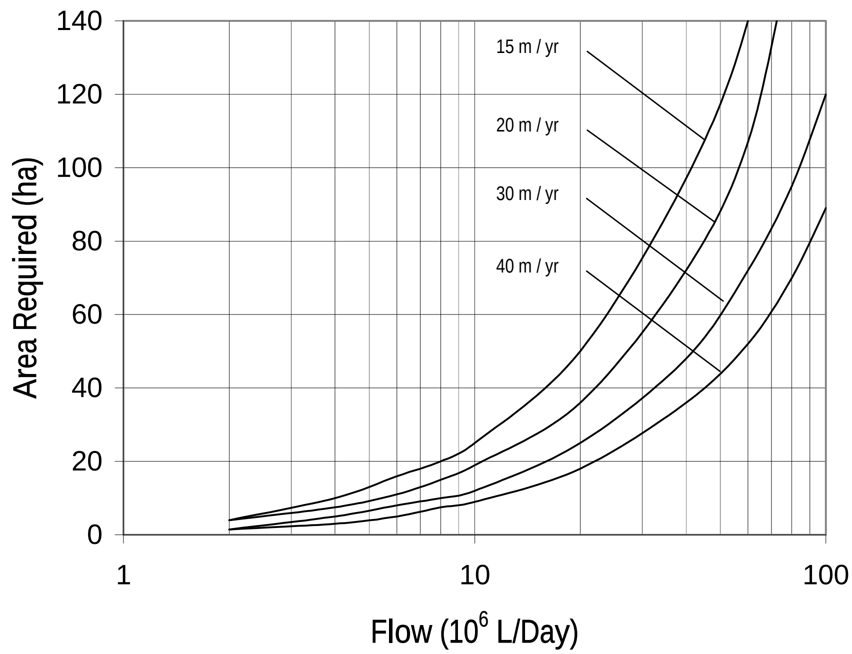
$$A_i = \frac{(Q\ \ell / d)(365d / yr)}{(L_w m / yr)(1000\ \ell / m^3)(10{,}000 m^2 / ha)}$$

If seasonal treated effluent flows are not equalized, the highest average seasonal flow rate should be used for design. The initial estimate of required land area computed using the equation above may be adjusted depending on constraints, as discussed in the section dealing with the layout of the infiltration area.[2]

d. Maximum daily infiltration capacity of the site in question can be read off the x-coordinate (Figure 10.1).

Figure 10.1

Determination of Land Area Required
For Rapid Infiltration Systems



The infiltration rate must be confirmed by field testing. Surface area requirements for an RI system must include:

a. infiltration basins and dykes;

b. maintenance and laboratory buildings(s);

c. possibly on-site treatment facilities;

d. on-site roads;

e. expansion and emergency use areas;

f. buffer strips.

10.2.3.3 Loading Cycle

In Atlantic Canada RI systems would likely require an altered loading cycle with respect to seasons because longer resting periods may be required for soil drying and aeration during winter. Decreasing the application rate and increasing the length of the application and resting period are possible means of overcoming the problems of winter application.

Suggested loading cycles are shown in Table 10.3. The values given in this table are considered guidelines. Actual loading cycles should take into account site-specific conditions.

TABLE 10.3 – SUGGESTED HYDRAULIC LOADING CYCLES FOR RAPID INFILTRATION SYSTEMS

Objective of Preapplication Treatment Period	Season	Application Period (Days)	Drying (Days)
Maximize infiltration rates of nitrification	Summer	1 - 3	4 - 5
	Winter	1 - 3	5 – 12
Maximize nitrogen removal	Summer	7 - 9	10 - 15
	Winter	9 - 12	12 - 18

10.2.3.4 Application Rate

Once the loading rate and loading cycle have been established, the application rate can be calculated. For example, if the hydraulic loading rate is 20 m/annum and the loading cycle is one day of application alternated with seven days of drying, the application rate is as follows:

$$A = B \times \frac{(C + D)}{C} \times E$$

A = Daily Application Rate
B = Hydraulic loading rate
C = Time on
D = Time off
E = Conversion Factor (Annual to Daily)

The application rate should be used to determine the maximum depth of the applied treated effluent. For instance, if the measured basin infiltration rate is 1.7×10^{-4} cm/s the maximum wastewater depth will be the daily application rate minus 1.7×10^{-4}.

In general, maximum treated effluent depth should not exceed 50 cm with a preferable maximum depth of 30 cm. If the treated effluent depth calculation indicates the recommended maximum will be exceeded, either the loading rate should be decreased or the loading cycle adjusted until the maximum basin depth is acceptable.

10.2.3.5 Monitoring

A monitoring program should provide applied treated effluent quality, the quality of groundwater affected by the RI system and, if required, an analysis of the soil affected by the RI system. Several groundwater samples should be collected from sites expected to be influenced by RI and compared with samples from areas not affected by treated effluent infiltration.

10.2.3.6 Separation Distances

The requirements for RI separation distances are outlined in Section 10.3.3.2.

10.2.4 Runoff

The system shall be designed to prevent surface runoff from entering or leaving the project site.

10.2.5 Fencing and Warning Signs

The project area shall be enclosed with a suitable fence to exclude livestock and discourage trespassing, depending on the level of treatment provided and type of effluent disposal used. A vehicle access gate of sufficient width to accommodate mowing equipment should be provided. All access gates should be provided with locks.

Appropriate signs should be provided along the fence around the project boundaries where necessary to designate the nature of the facility and advise against trespassing.

10.3 GUIDELINES FOR TREATED EFFLUENT IRRIGATION

Treated municipal effluent does not always meet a quality standard that would enable its unrestricted discharge to the sensitive receiving environment. For land application, concerns still remain with respect to elevated concentrations of soluble salts, nutrients, and microbiological quality of the treated effluent.

The major difference between municipal treated effluent and “high quality irrigation water” is the higher concentration of living and nonliving organic material, nitrogen, phosphorus, and in some instances, higher sodium and salt levels in the municipal treated effluent Low concentration of grease, oil, detergents, and certain metals may also be present, but these are generally at concentrations that do not adversely impact crop’s and/or the land if applied through irrigation at rates compatible with a crops seasonal water deficit need. Treated effluent suitability for irrigation is based on a select set of water quality parameters to be tested prior to and during their release. Site acceptability is to be based on pertinent soil and geological properties, topography, hydrology, climate, and zoning and cropping intentions.

In contrast with natural irrigation waters, municipal treated effluent has numerous additional health and environmental factors that need to be evaluated to ensure no detrimental impacts occur ~~for~~ from their use. Due to the origin, the variety and the often changing quality of wastewater generated by municipalities, it is imperative that municipal treated effluent be tested for a much wider range of water quality parameters than is currently necessary for irrigation with natural waters. Irrigation with municipal treated effluent is a suitable disposal option in Atlantic Canada where additional moisture can be effectively utilized for improved crop production. Treated effluent loading is to be based on the consumptive water use of the crop being grown. This loading, however, must also consider seasonal moisture deficiencies, system application efficiencies, and additional considerations related to annual soil leaching and crop nutrient utilization factors. The primary objective should be enhancement of crop production. The root zone of productive soils can often serve as one of the most active media for the decomposition, immobilization, or utilization of wastes. Considering these active processes in the topsoil, treated effluent can often be safely released to land at water quality standards less restrictive than those that would apply to a surface water release option. Further, with the added benefits currently applied to waste re-utilization processes and water conservation practices, treated effluent irrigation is considered an attractive waste disposal option.

10.3.1 ASSESSMENT OF MUNICIPAL EFFLUENT QUALITY FOR TREATED EFFLUENT IRRIGATION DEVELOPMENT

As water quality standards for municipal treated effluent discharging to surface water bodies become more stringent, the associated treatment costs correspondingly escalate. Irrigation is therefore becoming a more desired alternative for treated effluent disposal for many communities. However, since different water quality variables need to be considered when evaluating wastewater treatment plant effluents as a potential irrigation water source, than those considered for its direct discharge into a receiving stream, a specific set of treated effluent quality reporting requirements must be outlined and defined. In this overall treatise it is therefore important to first evaluate restrictions that may apply to the use of standard sources of irrigation water and then consider what supplemental evaluations would apply to treated effluent irrigation use.

10.3.1.1 Natural Irrigation Water Quality Characterization

The use of waters for irrigation application normally involves evaluation of the following water quality parameters:

- ***Electrical conductivity (EC)*:** is a reliable indicator of the total dissolved solids (salts) content of the water. The addition of irrigation water to soils adds to the concentration of salt in the soil. Concentration of these salts will result in an increase in osmotic potential in the soil solution interfering with extraction of water by the plants. Toxic effects may also result with an increase in salinity. EC is measured in dS m^{-1}. For specific values on acceptable EC levels in waters used for irrigation, refer to Table 10.4 that follows.

- ***Sodium Adsorption Ratio (SAR):*** is an indicator of the sodium hazard of water. Excess sodium in relation to calcium and magnesium concentrations in soils destroys soil structure that reduces permeability of the soil to water and air. Sodium may be toxic to some crops.

$$SAR = \frac{NA^{+}}{\sqrt{\frac{Ca^{2+} + Ma^{2+}}{2}}} \qquad \text{AND} \qquad SAR = \frac{NA^{+}}{\sqrt{Ca^{2+} + Mg^{2+}}}$$

(for concentrations in me/L) (for concentrations in mmole/L)

Cations are expressed in mequivalent of charge per litre or mmoles of charge per litre.

For specific values on acceptable SAR levels in waters used for irrigation, refer to Table 10.4.

- ***Boron (B):*** is very toxic to most crops at very low levels. In most jurisdictions, excess natural boron in soils and water has not been a problem. Acceptable boron concentrations for agricultural use are included in the applicable sections of the most recently published Canadian Environmental Quality Guidelines.

- ***Bicarbonate (HCO3):*** is considered hazardous when concentrations are excessive in some areas and not in others. Waters of high bicarbonate concentrations have been used for many years with no adverse effects in some jurisdictions. Acceptable bicarbonate concentrations for agricultural use are included in the applicable sections of the most recently published Canadian Environmental Quality Guidelines.

For further information on any other chemical parameters that may impact irrigation suitability from natural water sources, reference should be made to the applicable sections of the most recently published Canadian Environmental Quality Guidelines.

In light of the preceding factors, only two parameters, SAR and EC are normally of concern when irrigating with most available water sources in most jurisdictions. The limits for these parameters are as follows:

TABLE 10.4 IRRIGATION WATER QUALITY STANDARDS			
	Safe	Possibly Safe	Hazardous
EC dS m^{-1}	< 1.0	1.0 – 2.5	> 2.5
SAR	< 4	4 – 9	> 9

The limits under the heading “Safe”, are considered safe for all conditions. The “Possibly Safe” limits are considered safe for some conditions. Decisions should be based on the advice of a specialist. The “Hazardous” limits are considered unsuitable for almost all conditions.

Conditions to be assessed when dealing with waters that are “Possibly Safe” are as follows:

- Climate of the area - The deficit dictates the amount of water applied and consequently the amount of salt applied.
- Crops - Crops with high consumptive use require more irrigation water which again results in higher salt applications.
- Irrigation Practices - frequent irrigation results in less leaching than less frequent water applications. Light, frequent irrigation results in more evaporation. Fall irrigation results in increased leaching.
- Internal drainage - Good internal drainage facilitates rapid leaching of salts out of the root zone. System designs for irrigation with possibly safe water quality require specific investigation and the services of a specialist.

10.3.1.2 Comprehensive Treated Effluent Characterization

In contrast with fresh irrigation water, municipal treated effluent has additional health and environmental factors that need to be considered to ensure no detrimental impacts occur from its use. Due to the origin, variety and often changing quality of treated effluent generated by municipalities, towns and private sources, it is imperative that municipal treated effluents be periodically tested for a much wider range of water quality parameters than is currently necessary for irrigation with fresh waters. A comprehensive characterization of the treated effluent is necessary as part of the initial treated effluent irrigation application process and subsequently as may be specified by the regulatory agency having jurisdiction. Annual monitoring of a number of key biological and chemical indicator parameters, both prior to and subsequent to any treated effluent irrigation, should also be performed. The comprehensive treated effluent quality characterization requirements and the annual treated effluent quality monitoring requirements are discussed further in subsequent sections that follow.

The comprehensive characterization of treated effluent quality provides a means to ensure a basic level of irrigation quality control. It also provides useful baseline information to evaluate impacts from future irrigation. These impacts may relate to changes that occur in community water sources, waste treatment processes, community size, and community or industrial discharge loadings. In addition, the treated effluent quality characterization process may also provide an opportunity for community planners and engineering consultants to better evaluate the effectiveness of the treatment process and its ability to eliminate harmful constituents that could normally restrict the potential for irrigation use. The requirement of a comprehensive testing analysis in the initial application may enable future analytical testing requirements to be less onerous while still ensuring adequate protection of human health and the environment.

10.3.1.2.1 General Health Related Aspects

Biological assessment of municipal treated effluent is obtained by means of biological counts performed on the treated effluent prior to or on release. Potential human pathogens of concern found in domestic wastewater may be grouped into the following four categories:

- Bacteria (Salmonella, Shigella, Mycobacterium, Klebsiella, Clostridium)
- Protozoan parasites (Entamoeba, Giardia, Trichomonas)
- Helminth parasites (Ascaris, Toxacara, Taenia, Trichuris, Enterobius)
- Viruses (Picornaviruses, Adenoviruses, Rotaviruses)

The types and numbers of pathogenic organisms in wastewater depend on the nature of the wastewater being treated and the type of wastewater treatment. Wastewater organisms such as bacteria and viruses that are adsorbed to particulate matter tend to co-precipitate during settling phases of sewage treatment and, are thereby partly removed as solids from the water phase (Moore et al 1975). Similarly, encysted and egg stages of parasites, with specific gravities 1.06 to 1.2 (Englebrecht 1978), are effectively removed from the liquid wastewater during the settling phases of wastewater treatment process. The use of trickling filters, activated sludge systems, and effluent disinfection are additional treatment processes traditionally used to further reduce certain pathogenic organisms in wastewater. However, there is no single wastewater treatment process which will remove all pathogenic microorganisms. Many potentially disease causing microorganisms will therefore continue to exist in wastewater. The types and amounts of these microorganisms will vary greatly with the treatment process or combination of the processes utilized. Therefore for wastewater irrigation to be authorized, the minimum treatment requirement is secondary treatment followed by disinfection, and storage as required by regulatory agencies having jurisdiction. Despite their presence, the potential health hazard associated with utilizing treated effluent for irrigation can be minimized by adopting certain precautions and procedures.

The majority of the potentially harmful microorganisms are killed over a period of time by exposure to strong sunlight, high temperatures, and dry weather that may allow their direct application for sites with restricted access. Disinfection of treated effluent prior to land application shall be required where warranted by public health concerns, e.g. golf courses, parks, etc. Bacteriological quality shall meet the standards outlined in Table 10.6.

The timing of effluent irrigation with respect to harvesting crops and grazing livestock is also a factor that must be addressed; for further details reference should be made to Section 10.3.3.4.

Assessment of bacteriological constituents for the comprehensive treated effluent characterization requires only the testing of e-coli and/or fecal coliforms. Additional testing for other bacteriological parameters has not been found to be necessary in some jurisdictions as adoption of a best practicable treatment approach requiring primary treatment, storage, and various crop restrictions before irrigation, has proven appropriate in protecting the public from any adverse exposures to these particular constituents.

10.3.1.2.2 Other Water Quality Aspects

Other water quality aspects to be included in the comprehensive treated effluent characterization assessment prior to the development of a treated effluent irrigation system are included in the following section:

1) *General Chemical Parameters*

The general parameters are those that are analyzed to assess the effectiveness of the wastewater treatment process and to evaluate variability in the quality of the wastewater prior to its release to the environment. They also represent water quality values that, if exceeded, can often restrict treated wastewater sources from being considered for irrigation purposes.

- *Biochemical Oxygen Demand (BOD)* typically below 25 mg/L for most municipal treated effluents following secondary treatment.
- *Total Suspended Solids (TSS)* typically below 25 mg/L for most municipal treated effluent following secondary treatment
- *Chemical Oxygen Demand (COD)* typically below 50 mg/L for most municipal treated effluents following secondary treatment.
- *pH* typically ranges from 6.5 to 8.5 for most municipal treated effluents. These values are comparable to most natural surface waters and are considered to pose no restriction to irrigation use. A continued long-term use of waters outside this pH range could eventually alter naturally occurring pH levels in surface soils to which they are applied and therefore could possibly lead to micro nutrient imbalances and potential future crop production and fertility problems.
- *Electrical Conductivity (EC)* these values range widely within municipal treated effluent and like some natural water sources exceed levels that would be recommended for irrigation.

Those municipal treated effluent with EC values less than 1.0 dS/m are considered of good quality and should pose no problems for irrigation use, unless the sodium adsorption ratio (SAR) of the treated effluent is greater than 4.

Municipal treated effluents found to have EC values between 1.0 and 2.5 dS/m are considered marginal for irrigation and are usually restricted to use on land with favourable internal drainage properties. Crops normally grown under irrigation with such municipal treated effluent would not be impacted significantly. For situations where treated effluent of this quality is utilized for irrigation on a regular ongoing basis, supplemental approval conditions, requesting the periodic testing and reporting of salinity levels for lands being irrigated, would most likely apply. Results from such testing should be reported to the regulatory agency having jurisdiction, if:

- complaints of adverse impacts to the irrigated lands have been raised; or

- an application for approval renewal was being processed and concerns over deteriorating crop conditions were an issue.

Provision for periodic salt leaching is often advisable when considering treated effluent irrigation with water in this EC range.

Treated effluents with EC values exceeding 2.5 dS/m must not be used for irrigation purposes. Any such application would be restricted to a low volume discharge situation and require supplemental monitoring and reporting to be compiled on a regular basis.

It may be noted that EC values are often high in communities that utilize groundwater as a water supply source. Improving the quality of water supplies for these communities or adopting an alternate water supply source can lead to improvement in final treated effluent EC levels for these communities and possibly improve its suitability for irrigation.

- *Sodium Adsorption Ratios (SAR)* values can vary widely within municipal wastewater treatment facilities and like many natural water sources can often occur at levels that restrict their use for irrigation applications. Since adverse effects from high SAR are also dependent on the associated EC levels of the treated effluent, one should be aware of this interrelationship when evaluating SAR.

As a general guide treated effluent having SAR values less than 4 pose no problem for irrigation use.

Municipal treated effluents with SAR values ranging between 4 and 9 are considered marginal for irrigation and must include careful management to avoid potential damage to the land base or reduced crop productivity. Applying treated effluent of this quality can be particularly damaging on very fine textured soils or in situations where EC values of the treated effluent are less than 1 dS/m. Occasional calcium nitrate or gypsum applications may be helpful as a supplemental management practice on lands receiving irrigation applications of this quality for long periods of time. For situations where marginal municipal treated effluent quality is utilized for irrigation, supplemental approval conditions, including periodic testing would apply. Results from such testing should be reported to the regulatory agencies having jurisdiction, if:

- complaints of adverse impacts to the irrigated lands have been raised; or

- an application for approval renewal was being processed and concerns of deteriorating soil quality or reduced crop productivity were an identified issue.

Treated effluent with SAR values exceeding 9 should not be used for irrigation.

Communities using ion-exchange process for water softening can significantly increase SAR values in the wastewater. Hence, careful and regular monitoring of SAR levels within systems where water softeners are used is important.

2) Nutrients

One of the main advantages of using treated effluent irrigation is that it may often enhance the fertility of the lands to which it is applied. This can add considerably to potential crop yield and therefore the associated agricultural resource value. Nutrient loading rates, while significant, are seldom at levels that would present a concern when using municipal treated effluent for irrigation. Most nutrient levels are well within the range that can be assimilated by plants if the treated effluent is applied at a rate and frequency that conforms to active crop growth. Potential contamination of groundwater would only be a concern under extremely shallow groundwater levels, unsuitable soil conditions, or gross mismanagement of the applied treated effluent. Since all these factors are carefully considered as part of the guidelines, potential contamination of the groundwater should not present a concern. The following nutrients should be analyzed and reported as part of the comprehensive treated effluent quality characterization process:

a) Nitrogen can be evaluated in a number of different forms. Regular evaluation of nitrogen by analyzing for NO_3-N, NH_3-N, NO_2-N, and TKN should be conducted. The typical concentration for total nitrogen of most municipal treated effluent is up to 20 mg/L. This means that if 30 cm/yr of treated effluent were applied, an N loading of 30 to 60 kg/ha/yr. would be applied to the land base. Providing treated effluent is not applied in quantities that exceed the field moisture capacity during periods of treated effluent applications, and is applied during the active crop growing season, such loadings can be easily assimilated by the growing crop without harmful health or environmental concerns developing. Treated effluent that consists of a total nitrogen concentration within the typical range can easily be assimilated by the growing crop without harmful health or environmental concerns provided treated effluent is not applied in quantities that exceed the field moisture capacity and it is applied during active crop growing season.

b) Phosphorus is to be evaluated as total phosphorus. The typical concentration of total phosphorus in municipal treated effluent following secondary treatment is up to 6 mg/L. If 30 cm/yr of treated effluent were applied, this would translate to a P loading of 6 to 18 kg/ha/yr. Since these levels are considered to be reasonably low and phosphorus is effectively immobilized in most soils at shallow depths, the potential for adverse impacts on groundwater quality is remote. Care must be exercised, however, to ensure treated effluent applications are applied at rates that do not exceed the infiltration capacity of the soils as high phosphorus levels in surface runoff and erosion sediments can create significant environmental concern if washed into neighbouring lakes, streams or other surface water bodies.

c) Potassium is another major nutrient present in treated effluent of value for crop production that should be evaluated. The typical concentration for potassium in most municipal treated effluent is up to 40 mg/L. If 30 cm/yr of treated effluent were applied this would translate to a K loading of 15 to 120 kg/ha/yr. Such levels are normally assimilated by crops and are thus not considered to be an environmental or health risk.

3) Major Cations and Anions

The treated effluent should be analyzed and reported for the following cations and anions in the required comprehensive treated effluent characterization:

Calcium (Ca) mg/L	Magnesium (Mg) mg/L
Sodium (Na) mg/L	Carbonate (CO_3) mg/L
Bicarbonate (HCO_3) mg/L	Alkalinity, total ($CaCO_3$) mg/L
Fluoride (F) mg/L	Sulphate (SO_4) mg/L
Chloride (Cl) mg/L	

4) Metals

Uptake of harmful amounts of toxic heavy metals by plants is not considered a potential risk in use of municipal treated effluent, as most metals are removed from the wastewater in the primary treatment process. However as a precautionary measure, all wastewater should be initially tested for the following metals in Table 10.5 levels are below recommended CCME Canadian Environmental Quality Guidelines prior to granting authorization for irrigation application.

Since collection of this information is intended more as a general treated effluent quality characterization inventory rather than for purposes of assessing irrigation water quality limits, specific values will likely not be exceeded for most municipal treated effluent tested.

In addition, a careful evaluation of any industrial discharges into the municipal system and their potential impact on overall wastewater quality must also be addressed. If, due to the nature of these industrial activities, concerns relating to any other chemicals become evident, then these chemicals should also be added to the comprehensive list of suggested chemical parameters for treated effluent characterization.

TABLE 10.5 – Canadian Environmental Quality Guidelines for the Protection of Agricultural Water Uses		
Parameter	**Concentration (µg/L)[a]**	**Remarks[b]**
Aluminum	5000	Can cause non-productively in acid solids (pH < 5.5), but more alkaline solids at pH > 5.5 will precipitate the ion and eliminate any toxicity.
Arsenic	100	Toxicity to plants varies widely, ranging from 12 mg/L for Sudan grass to less than 0.05 mg/L for rice.
Beryllium	100	Toxicity to plants varies widely, ranging from 5 mg/L for kale to 0.5 mg/L from bush beans.
Boron	500-6000	Boron is very toxic to most crops at very low levels. In most jurisdictions, excess natural boron in soils and water has not been a problem.
Cadmium	5.1	Toxic to beans, beets and turnips at concentrations as low as 0.1 mg/L in nutrient solutions. Conservative limits recommended because of its potential for accumulation in plants and soils to concentrations that may be harmful to humans.
Chromium - Trivalent Cr (iii) - Hexavalent Cr (vi)	4.9 8.0	Not generally recognized as an essential growth element. Conservative limits recommended because of lack of knowledge on toxicity to plants.
Cobalt	50	Toxic to tomato plants at 0.1 mg/L in nutrient solution. Tends to be inactivated by neutral and alkaline soils.
Copper	200-1000	Toxic to a number of plants at 0.1 to 1.0 mg/L in nutrient solutions.
Fluoride	1000	Inactivated by neutral and alkaline soils.
Iron	5000	Not toxic to plants in aerated soils but can contribute to soil acidification and loss of reduced availability of essential phosphorus and molybdenum. Overhead sprinkling may result in unsightly deposits on plants, equipment, and buildings.
Lead	200	Can inhibit plant cell growth at very high concentrations.
Lithium	2500	Tolerated by most crops up to 5 mg/L; mobile in soil. Toxic to citrus at low levels (>0.075 mg/L). Acts similar to boron.
Manganese	200	Toxic to a number of crops at a few

TABLE 10.5 – Canadian Environmental Quality Guidelines for the Protection of Agricultural Water Uses

Parameter	Concentration (µg/L)[a]	Remarks[b]
		tenths mg to a few mg/L, but usually only in acid soils.
Molybdenum	10-50	Not toxic to plants at normal concentrations in soil and water. Can be toxic to livestock if forage is grown in soils with high levels of available molybdenum.
Nickel	200	Toxic to a number of plants at 0.5 to 1.0 mg/L; reduced toxicity at neutral or alkaline pH
Selenium	20-50	Toxic to plants at concentrations as low as 0.025mg/L and toxic to livestock if forage is grown in soils with relatively high levels of added selenium. An essential element for animals bit in very low concentrations.
Tin	-	Effectively excluded by plants; specific tolerance unknown.
Titanium	-	(See remark for tin)
Tungsten	-	(See remark for tin)
Uranium	10	
Vanadium	100	Toxic to many plants at relatively low concentrations
Zinc	1000-5000	Toxic to many plants at widely varying concentrations; reduced toxicity at pH > 6.0 and in fine-textured or organic soils.

a- (Limits are adopted from the Summary Table, Canadian Environmental Quality Guidelines, Canadian Council of Minister of the Environment, 2005)

b- Adopted from Metcalf & Eddy Inc., "Wastewater Engineering: Treatment and Reuse", 2003.

10.3.1.3 Annual Treated Effluent Quality Monitoring Requirements

Wastewater must also be analyzed and results reported annually for certain water quality parameters, both prior to and on completion of each irrigation application event. This monitoring requirement is in addition to the comprehensive treated effluent characterization outlined in Section 10.3.1.2. For annual testing purposes the treated effluent should be sampled at the pipe inlet of the irrigation distribution equipment. The treated effluent quality for treated effluent irrigation shall meet the standards specified in Table 10.6.

TABLE 10.6 – Treated Effluent Quality Guidelines for Wastewater Irrigation				
Parameter	**Guidelines**		**Type of Sample**	**Comments**
	Restricted Use	**Unrestricted Use**		
E.Coli*	<200MPN/100 mL	<2MPN/100 mL	Grab(twice/month)	For unrestricted use, sampling should be conducted prior to startup and on a weekly basis. For restricted use sampling should be conducted prior to startup and then one more sample sometime during discharge.
BOD	25 mg/L	10 mg/L	Grab/composite**	Sampling should be conducted at startup and once during discharge.
COD	50 mg/L	20 mg/L	Grab/composite**	Samples collected twice annually, prior to and on completion of a major irrigation event
TSS	25 mg/L	10 mg/L	Grab/composite**	For unrestricted use, sampling should be conducted prior to startup and on a weekly basis. For restricted use sampling should be conducted prior to startup and then one more sample sometime during discharge.
Electrical Conductivity	1.0 - 2.5 dS/m	<1.0 dS/m	Grab/composite**	Samples collected twice annually, prior to and on completion of a major irrigation event
SAR	4-9 for when EC >1.0 dS/m	<4	Grab/composite** (once/month)	Samples collected twice annually, prior to and on completion of a major irrigation event
pH	6.5 to 8.5		Grab/composite**	Samples collected twice annually, prior to and on completion of a major irrigation event

* For golf courses and parks only.
** Grab sample would suffice if storage is provided; Composite sample is required if storage is not provided.
MPN – Most Probable Number

10.3.1.4 Other Requirements

TABLE 10.7 GUIDELINES FOR INTERPRETATIONS OF WATER QUALITY FOR IRRIGATION[a]

Potential irrigation problem	Units	Degree of restriction on use		
		None	Slight to moderate	Severe
Salinity (affects crop water availability)				
EC_w	dS/m or mmho/cm	< 0.7	0.7 - 3.0	> 3.0
TDS	mg/l	< 450	450 - 2000	> 2000
Permeability (affects irrigation rate of water into the soil. Evaluate using EC_w and SAR or adj R_{Na} together)[b]				
adj R_{Na} = 0 - 3		and $EC_w \geq 0.7$	0.7 – 0.2	< 0.2
3 – 6		≥ 1.2	1.2 -0.3	< 0.3
6 – 12		≥ 1.9	1.9 – 0.5	< 0.5
12 – 20		≥ 2.9	2.9 – 1.3	< 1.3
20 – 40		≥ 5.0	5.0 – 2.9	< 2.9
Specific ion toxicity (affects sensitive crops):				
Sodium (Na)				
Sprinkler irrigation	mg/l	< 70	> 70	
Chloride (Cl)				
Sprinkler irrigation	mg/l	< 100	> 100	
Boron (B)	mg/l	< 0.7	0.7 – 3.0	> 3.0
Trace Elements (See Table 10. 5)				
Miscellaneous effects (affects susceptible crops):				
Nitrogen (total – N)	mg/l	< 5	5 - 30	> 30
pH	Unit		Normal range 6.5 – 8.4	

a – From Metcalf & Eddy Inc., "Wastewater Engineering: Treatment and Reuse", 2003.
b – For treated effluent irrigation, it is recommended that SAR be adjusted to include a more correct estimate of calcium in the soil water.

10.3.2 ASSESSMENT OF LAND SUITABILITY FOR PROPOSED TREATED EFFLUENT IRRIGATION DEVELOPMENT

Land classification and other relevant soil, climate, and groundwater assessment activities are generally performed after completing the comprehensive treated effluent characterization assessment, and results of the treated effluent characterization have shown that the wastewater is suitable for irrigation.

Careful assessment and characterization of the land base including associated soil, groundwater, and other crop related inputs are required prior to proceeding with actual design of the treated effluent irrigation system. A site is classed as suitable for treated effluent application only if it is found to possess soil, climatic, and physical characteristics that enable effective utilization of the treated effluent applied without causing future damage to the land base or to the underlying groundwater. Site conditions must also be such that they effectively restrict any detrimental offsite movement of the treated effluent through leaching, groundwater migration, surface runoff, or drift from irrigation spray. The following sections outline land classification, soil, and other testing requirements that must be addressed prior to actual development of an applicable treated effluent irrigation system design and issuance of the authorized approval.

10.3.2.1 Site Suitability

Before treated effluent irrigation development can proceed, the lands to be irrigated must first be reviewed and approved by the regulatory agencies having jurisdiction.

For purposes of the regulatory review, the following information shall be provided:

- a map showing the location of all soil sampling and description sites and surrounding activities or uses;
- a copy of all soil logs;
- a copy of soil chemical and physical analysis completed for the classification;
- a legible soil map that shows the soil description for the affected areas;
- a drafted land classification map at a scale of 1:5000 showing the land class symbol, drainability and limitations for each unit classified; and
- a remark sheet or report that accompanies the land classification map. The typed report shall briefly describe each land class unit with regard to the type of soils, soil texture, irrigation suitability, suitability for gravity or sprinkler irrigation development, the limitations of the irrigable units and reasons why nonirrigable units are rated nonirrigable. A statistical summary table that shows the following, where applicable, shall also be included: total irrigable acres; total nonirrigable acres; right-of-way and easement acres; not investigated acres; and acres of farmsteads or other physical features that are present.

Municipal treated effluent have much higher nitrate levels than other irrigation water sources. It is therefore necessary to further restrict treated effluent application on lands where the natural water table is less than 1 m below ground surface and/or impermeable bedrock or other geological barriers exist at less than 1 m below ground surface.

The following soil and site characterization details must also be collected and reported, in addition to completing the required land classification designations and mapping.

10.3.2.1.1 Soil Assessment[3]

Soil assessment involves examination of test pits and testing of soil permeability.

1) **Test Pits**

Test pits provide information about the soil profile at the proposed location of the irrigation system. This information must include the following:

- Organic layer
- Total soil depth
- Effective soil depth
- Total depth of test pit
- Root penetration
- Depth to bedrock
- Depth to layer of soil with unacceptable permeability
- Determination of highest seasonal water table
  - Presence and depth of mottling
  - Depth to water
  - Moisture content (saturated, moist, dry, etc.)
  - Perched water table
- Soil profile:
  - Description of soil (including all soil from unacceptably high to unacceptably low)
  - Depth of each layer
  - Texture of soil
  - Moisture content (saturated moist, dray, etc.)
  - Density (loose, medium, compact, tight)
  - Colour
  - Structure

For safety, the pit should be more than 1.2 m deep, with sloping sides and an entrance ramp for easy access and escape in the event of a soil slide. All soil removed from the pit should be placed a minimum of 1 m from the edge of the pit. If the pit is dug by backhoe and verification of subsoil conditions is required, the pit may be taken to a greater depth, but inspection should be carried out from the surface with the aid of samples of soil recovered by the machine bucket. A soil profile can then be recorded based on the variation in soil characteristics with depth.

All test pits must be dug in compliance with the regulatory agency having jurisdiction.

2) **In-situ Permeability Tests**

In-situ permeability tests can be used to confirm the estimation of soil permeability based on the visual assessment of soil properties in the test pit. When using these tests to verify results, a minimum of three tests should be done. If the tests are not of similar order of magnitude, more tests should be conducted.

These tests may also be used for the:

- Determination of particular sandy gravel as a soil with acceptable or unacceptably high permeability.
- Determination of a particular soil as an unacceptably low or an impermeable soil.
- Confirmation of visual assessment of soils for higher flow systems, such as commercial and institutional buildings.

10.3.2.1.2 Soil Properties[3]

Some soil properties that are useful in assessing soil suitability include: Texture, structure, colour, density and depth.

A soils consulting engineer should assess the soil results and make recommendations.

10.3.2.1.3 Topography

Topographic features such as relief, site and shape of fields, soil type and texture, brush/tree cover, and surface drainage features must be evaluated for site suitability. Land may not be considered suitable for irrigation due to one or a combination of factors such as: steep slopes, hummocky relief, brush/tree cover, small or irregular shape, sloughs, wetlands, and rough broken topography.

The topography is to be classified as to its suitability for treated effluent irrigation. The topography at each site is also to be mapped. This topographic mapping should be provided at a level of detail not less than a scale of 1:10,000 and a contour interval of 0.5m. The information should be gathered either from a topographic survey of the land parcel or from a suitable scaled orthophoto or photogrammetric mapping of the property. This mapping must reference grid and property boundaries, treated effluent irrigation development boundaries, and soil test and groundwater test site locations. Inclusion of recent stereoscopic air photo coverage at a 1:10,000 scale would be advisable, but is not a requirement.

10.3.2.2 Other Requirements

Other information in the initial site assessment process must include:

a) Location and mapping of any surface water courses, water bodies, or domestic wells located on or within 150m of the treated effluent development site.
b) Location and mapping of any residential dwelling on or within 400m of irrigation sites and 150m of infiltration sites.
c) Location and mapping of all public roads, highways, or other public corridors on or within 30m of the treated effluent development site.

These site-specific requirements are intended to provide baseline information on all sites to be developed for treated effluent irrigation purposes. The knowledge is intended to assist in evaluating potential impacts of long-term treated effluent irrigation on the land base over time.

10.3.3 ASSESSMENT OF SYSTEM DESIGN NEEDS FOR PROPOSED TREATED EFFLUENT IRRIGATION DEVELOPMENT

Treated effluent irrigation system design is undertaken once water quality assessment and land suitability assessment are affirmed. The design integrates treated effluent quality with land base limitations and restrictions that relate to cropping, climate, application, and public acceptance issues. The overall design includes an account of the following:

10.3.3.1 Climate

There are a number of climate factors that must be considered to ensure an effective treated effluent irrigation system design. These factors are defined as follows:

- Adequate storage must be provided for periods when treated effluent can not be disposed of by irrigation due to unauthorized periods or climate condition.

- wind speeds are in excess of 30 km/hr, or during periods of intense or prolonged precipitation.

- Seasonal mean precipitation, evapotranspiration and seasonal crop moisture demands must therefore be established for the infiltration or for the irrigation period authorized and be applicable to the geographical area of the specific project. These requirements will be necessary to determine the land base required to effectively dispose of the annual volumes of community wastewater available for discharge. Sufficient land to handle this anticipated flow must be obtained. Irrigation systems should be designed to have an almost complete utilization of nutrients and about 85% utilization of water. Since annual values will vary from year to year, design must allow for either a 25% treated effluent storage carry over or provision for an occasional expansion in irrigation system and land base design in order to accommodate the lower treated effluent irrigation discharge allotments required during wet years. Provision for supplemental irrigation sources in dry periods may also be considered.

10.3.3.2 Land Area

There are a number of land-related factors relevant to irrigation system design that must be considered. These factors are defined as follows:

- Specific irrigation design features must be provided that will avoid application of irrigation treated effluent to any non-irrigable land areas (greater than 15 percent of the area to be irrigated).
- The amount of land and equipment required will depend upon the mean annual consumptive use of water by plants, natural precipitation from April through September, an irrigation efficiency factor, and an appropriate leaching requirement. If no provisions are provided for extra treated effluent storage during abnormally wet years, additional land areas and equipment will be required to meet these needs.
- The land area to be accumulated must also allow for any buffer zones or setback limits that apply on or around land areas where treated effluent

irrigation is to be undertaken. Set backs and buffer zones that will apply are outlined in Table 10.8.

TABLE 10.8 SETBACK REQUIREMENTS	
Parameter	**Requirements**
Adjacent Properties	Buffer Zone of 15 M between irrigated land and adjacent property owners.*
Adjacent Dwellings	Buffer Zone of a minimum of 50 M between irrigated land and any occupied dwellings.*
Public Rights of Way	Buffer Zone of a minimum of 25 M between irrigated land and any public right of way.
Potable Water Wells	Buffer Zone of a minimum of 30 M between irrigated land and any potable water well.
Watercourses, Rivers, Streams, etc.	Buffer Zone of a minimum of 20 M between irrigated land and any Watercourse.**

* Distance maybe reduced with the signed permission of adjacent property owner.

** Watercourses used for golf course irrigation area exempt from the buffer zone. Distance maybe reduced depending on the actual quality of irrigation water.

- In addition to the above consideration, the land area to be used for treated effluent irrigation and storage cells shall be sufficiently large such that treated effluent discharge will not occur during the following periods.

  i) Outside the growing season except if authorized for a fall irrigation.
  ii) During and for 30 days prior to the harvesting of crops.
  iii) During and for 30 days prior to grazing by dairy cattle.
  iv) During and for 7 days prior to pasturing by livestock other than dairy cattle.

- A plan illustrating the layout of the irrigation system designed to irrigate the site shall be provided. The plan must illustrate: the boundaries of the particular section(s) within which irrigation application will take place, the boundaries of the land area to which treated effluent will be applied, the extra land area to be irrigated during wet seasons when above average mean seasonal precipitation occurs if design for extra lagoon storage is not provided and the actual orientation of irrigation equipment, sprinkler head sizing, operating pressures and overall irrigation system layout.

10.3.3.3 Application Loading Rates

The rate of treated effluent application loading shall depend on individual crop moisture and nutrient uptake needs. These factors are defined as follows:

- Nitrogen is usually the only nutrient that may prove to be restricting in respect to the amount of treated effluent that may be applied in a given irrigation season. The amount of plant available nitrogen, based on amount of treated effluent that is applied, should be calculated and noted as kg per ha per year. As long as these rates do not exceed the annual crop nitrogen removal rates and an active crop-harvesting program exists no restrictions to the application of typical treated effluent should apply. Other major nutrients generally do not exceed annual crop uptake requirements and therefore do not pose a risk to water quality.

- Crop moisture requirements thus become the main determining factor in establishing acceptable treated effluent irrigation application limits. Annual treated effluent application amounts ultimately depend on the annual seasonal crop needs minus season rainfall. However, other factors such as: soil moisture holding capacity, soil infiltration rate, crop rooting depth, rate frequency and duration of irrigation event, irrigation system efficiency, and soil leaching requirements, will have a bearing on the efficiency of crop moisture utilization and therefore need to be evaluated as part of any irrigation system design. A local irrigation specialist or a qualified agricultural consulting firm should be consulted to ensure accurate assessments of these values for different locations. The eventual design of the irrigation system must ensure effective uniform application of the treated effluent and prevent any surface runoff or prolonged surface ponding to occur during application. The irrigation system must also be designed to avoid treated effluent applications that exceed crop seasonal water deficit requirements and leaching demands. If natural precipitation during the irrigation off-season is not sufficient to enable leaching of excess salt accumulations, the irrigation system must account for an annual leaching factor of 10 percent. This being required to assist with flushing excess soluble salts below the crop root zone.

10.3.3.4 Crop Considerations

Only certain crops are deemed suitable for production on lands to be irrigated with treated effluent. Crops for direct human consumption are not suitable for effluent irrigation or infiltration. The current authorized crops include only forages, coarse grains, turf, trees and oil seeds. See section 10.3.1.4 for other considerations.

10.3.3.5 Treated Effluent Storage Ponds

The design of any storage reservoir required to retain treated effluent during periods of restricted irrigation must meet current design criteria as described in Chapter 7. If the pond is designed to hold treated effluent then it is not required to have an impermeable liner. Where odour problems may occur, aeration of the storage reservoir may be necessary.

10.3.4 SYSTEM OPERATION

Once the wastewater development project has been approved and constructed, an Approval to Operate is required before operation can proceed. The approval will spell out operating conditions and requirements for the system.

The municipality must be responsible for the proper operation of the irrigation project, even if someone other than the municipality is actually managing the system. Proper operation of the system is essential for longevity of the system, for a high degree of treatment and for high production. Although crop production is not the prime objective of the system, a vigorous crop growth is essential for utilization of water and nutrients.

Due to the great variation in waste concentration, soils, and climate, no attempt will be made to elaborate further on irrigation management. Specific operational requirements will be stated in the approval to operate.

Operating conditions and requirements for the system must be described prior to receiving approval. Due to the great variation in waste concentration, soils, and climate, no attempt will be made to elaborate further on irrigation management in this document. Specific operational requirements will be stated in the certificate of approval.

10.4 REUSE OF TREATED EFFLUENT FOR GOLF COURSE IRRIGATION

Treated effluent for golf course irrigation, where acceptable to the regulatory agencies having jurisdiction, is treated to the extent that it can beneficially be reused without adverse effects of public health or the environment. Benefits of the use of treated effluent for golf course irrigation include:

- A more cost effective and environmentally beneficial alternative compared to other methods of treated effluent disposal.
- Conservation of water resources.
- Reduced demand on municipal water supply.
- Addition of nutrients and micronutrients is beneficial to turf growth.

Planning, design and management of golf course irrigation systems that use treated effluent must take the following into account:

- Regulatory concerns regarding protection of public health and the environment.
- Concerns about possible effects of treated effluent on golf course soils and vegetation.
- Cost associated with installation and operation of an irrigation system.

10.4.1 Environment

Positive environmental affects of irrigation with treated effluent include:

- Avoiding the need to discharge effluent into sensitive areas such as beaches or water supplies.
- Conservation of scarce water resources which are replaced by treated effluent.
- The nutrient content of treated effluent can provide an economic advantage by reducing the cost of commercial fertilizers.

Environmental concerns could include the following:

- Contamination of surface water and groundwaters by bacteria and other organisms.
- Odours associated with treated effluent may be noticeable to golfers.
- Nitrate contamination of groundwater supplies.
- Unsightly algal and weed growths in reservoirs, and ponds.

These concerns should be addressed by secondary treatment and disinfection of effluent that will be subjected to prolonged storage before it is reused for golf course irrigation.

Prolonged storage is expected to remove the slight musty smell of fresh secondary effluent, which might be noticeable and distasteful to some golfers.

Nitrogen and phosphorus are chemical nutrients that are applied as a part of turf grass management. These nutrients, usually provided by commercial fertilizers, may be replaced in part by use of treated effluent.

If nutrient application in commercial fertilizers or treated effluent is properly managed, there is little potential for unsightly and possibly odourous algal and weed growths in lakes, and ponds or of nitrate contamination of groundwater.

If nitrogen is applied at a rate that exceeds the ability of the plant and soil system to contain it or convert it to nitrogen gas, the excess nitrogen may pass through the surface soil and into groundwater. High concentrations of nitrogen, particularly in the form of nitrate, are considered a health hazard in drinking water.

If treated effluent is impounded in an open reservoir a water quality maintenance program which should include one or more of the following measures, is needed:

- Screening or filtration to remove solids, such as algal growths, to reduce maintenance of sprinkler systems.
- Control or prevention of algal growth by an algicide, or a light inhibitor such as blue dye.
- A mixing system.
- Rechlorination to maintain a residual in the distribution system.

Adequate circulation and aeration are necessary for algal and odour control. Aeration can be provided by fountains, air injection, waterfalls, or constructed wetlands.

Algae and weeds in ponds are concerns that can be addressed by assuring that nutrients in fertilizers or effluent are applied to or washed into, these bodies.

If nutrient application in commercial fertilizers or treated effluent is properly managed there is little potential for unsightly and possibly odourous algal and weed growths in lakes and ponds, or of nitrate contamination of groundwater.

If nitrogen is applied at a rate that exceeds the ability of the plant and soil system to contain it or convert it to nitrogen gas, the excess nitrogen may pass through the surface soil and into groundwater. High concentrations of nitrogen, particularly in the form of nitrate, are considered a health hazard in drinking water.

10.4.2 Soils and Vegetation

The quality of water that is applied in irrigation is an obvious concern to those responsible for management of the soils and vegetation on which a golf course depends.

Irrigation water quality parameters that are of concern include: pH, carbonate, bicarbonate, calcium, magnesium, sodium, potassium, conductivity, boron, chloride, sulphate, and adjusted SAR (Sodium Adsorption Ratio)
Bicarbonates and carbonates both increase pH, and are a source of alkalinity, which may affect the water and soil. If the total concentration of bicarbonates plus carbonates exceeds 150 mg/L the resulting increase in pH may affect nutrient availability. Options used to offset this effect include use of acid-forming nitrogen fertilizers, sulphur addition to the soil, or acid injection into the irrigation water.

The adjusted SAR is based on the ratio of sodium to calcium + magnesium in the water. Excess sodium replaces calcium on soil exchange sites, which can result in soil compaction and reduce infiltration into the soil. The usual recommendation for a SAR above 10 is application of calcium, usually a gypsum, and excess irrigation to leach the sodium.

Conductivity is a measure of total soluble salts, or salinity of the water. While most turf species used on golf courses are reasonably salt tolerant, ground covers, ornamental plants, trees and shrubs may be affected if salt concentrations are too high. There is no recommended restriction on use of irrigation water if the conductivity is less than 3 mmhos/cm.

Boron, chloride, and sulphate may be toxic to plants if concentrations are too high. Concentration of boron higher than 1 to 2 mg/L (0.33 for some ornamental plants), and of chloride plus sulphate above 250 to 400 mg/L, are considered excessive.

Other parameters that may require regular or occasional measurement are nitrogen, phosphorus, suspended solids, and heavy metals. An excess of inorganic or organic nitrogen may require careful control of fertilizer application. Phosphorus, and nitrogen, may alter the hydraulic properties of soils, or clog sprinkler head openings. If concentrations of heavy metals build up in soils they may complex with phosphorus and other elements and make them unavailable to plants.

10.4.3 Planning

The following is a developers/Operators checklist for use of treated effluent for golf course irrigation:

- Sampling soils: sample soils well in advance of conversion to effluent, to track effects of the change; sample from difference parts of the course - tees, greens, fairways, rough; sample irrigated soil quarterly to allow for adjustment of watering schedule and use of mitigative measures.
- Water Quality: initial and periodic water analysis; verify effluent source, noting that if industrial waste is included more undesirable elements may

be present; verify treatment type, more is better; establish maximum BOD, TSS and TDS levels in advance.
- Pumping and storage: considerations include – existing pumped or gravity supply need for additional pumping; form and amount of storage; need for algae control; possible use of fresh water for greens, tees, ornamental lakes and sensitive plants.
- Miscellaneous: operation, maintenance and safety issued to be considered.
- Signage
- Notification on score cards and that treated effluent is being used.
- Time of day of irrigation.

10.4.4 Design

Design considerations related to irrigation with treated effluent include:

- Screening and/or filtration of stored effluent to avoid clogging of the irrigation system.
- If acid injection is involves, consideration of corrosion effects.
- Avoidance of cross-connections between potable and non-potable water systems.
- Labelling and colour coding of non-potable pipes and equipment.
- Provision of flush valves at low spots and dead ends to allow removal of debris.
- Location of sprinkler heads to avoid contamination of drinking fountains, canteens, food and drink machines, etc.

Sprinkler head location may also have to consider contamination of, or nutrient addition to, water hazards.

Reasons for treated effluent storage include:

- To balance supply and demand
- To supplement treated effluent with other source
- To contain excess on potable water
- A combination of the above

Seasonal storage may be required where no alternative effluent disposal method is available.

10.4.5 Management Concerns

Issues that may concern a golf course superintendent include:

- Many older greens have low infiltration rates, and require close attention to avoid turf failure, the risk of which may be increased if there is a possibility that use of treated effluent may further reduce infiltration capacity and require reconstruction of greens.
- If the level of the nutrients in the treated effluent is high, and especially if it is variable, staff will lose the ability to carefully control rates of nutrient application.
- If the application of treated effluent only at night results in a shortened application period, application rates will be increased, and the capacity of pumps and piping may be inadequate.
- If the course is committed to accept and use a certain amount of treated effluent and there is no alternative use for or disposal of water in excess of that used for irrigation, the superintendent may be forced to overwater the

course, i.e., irrigation will be based on effluent disposal needs instead of proper golf course management.

Management functions include:

- Public relations: Member and player concerns that must be satisfied include odours, course appearance, legal liability, health risks, and adjacent property values.
- Design and Construction Administration: plan checking, inspection, record drawings.
- Operation and Maintenance: Monitoring and Testing: Staff, player, and public safety.

Footnote References

1. WEF, “Manual of Practice 11 – Operation of Municipal Wastewater Treatment Plants”, Volumes I, II and III, 1996.

2. Metcalf & Eddy Inc., "Wastewater Engineering: Treatment, Disposal, Reuse", 1991.

3. Nova Scotia Department of Environment, “Onsite Sewage Disposal Systems Technical Guidelines”, November 2005.