SUBCHAPTER F: ACTIVATED SLUDGE SYSTEMS

§§217.151 – 217.164

Statutory Authority

[Language drafted and provided for inclusion by OLS attorney assigned to this rulemaking project (this should be done simultaneously while the Fiscal Note information is being drafted (if not before)).

Note: The **1st paragraph** of a Statutory Authority should state what the rules are proposed "under the authority of," and the **2nd paragraph** should list (no titles) any bills, statutes (state or federal) the rules implement.]

RULE OF THUMB: for existing rules/sections, language must have been downloaded from 30 Texas Administrative Code as this is the *official* version of the rules.

- **NEW language**: to designate language that is *new* to 30 TAC, <u>you ***must*** underline new language that does *not* currently exist in TAC, including punctuation</u>
- **Delete existing language**: to designate existing language in 30 TAC that is *obsolete, no longer required/needed*, [you ***must*** place that language between brackets] in order to show deletion of that language from 30 TAC
- <u>new language</u> before [old language]

§217.151. Requirements for an Aeration Basin.

(a) Unless designed for advanced nutrient removal, an aeration basin must be designed to maintain a minimum dissolved oxygen concentration of 2.0 milligrams per liter (mg/l) throughout the aeration basin at the maximum diurnal organic loading rate determined in §217.32(a)(3) and Figure: 30 TAC §217.32(a)(3) of this title (relating to Organic Loading and Flows for New Wastewater Treatment Facilities).

(b) The volume of aerated influent wastewater channels and aerated mixed liquor transfer channels of an activated sludge system may be used to meet aeration basin volume requirements, provided the activated sludge system uses aeration by diffused air and the diffuser depth conforms to the requirements of §217.155(b)(5)(A) of this title (relating to Aeration Equipment Sizing).

(c) The use of a contact stabilization system for nitrification is prohibited.

§217.152. Requirements for Clarifiers.

(a) Inlets.

(1) A clarifier must have an inlet valve or gate.

(2) A clarifier inlet must provide uniform flow and stilling.

(3) A transfer pipe must not trap or entrain air.

(4) Vertical flow velocity through an inlet stilling well must not exceed 0.15 feet per second at peak flow.

(5) An inlet distribution channel must be designed to prevent the settling of solids in the channel.

(b) Pumped Inflow.

(1) For a wastewater treatment facility with pumped inflow, a clarifier must be able to accommodate all flow without overloading or overtopping the clarifier effluent weir.

(2) A clarifier must hydraulically accommodate peak flows without adversely affecting treatment in the clarifier or in subsequent treatment units.

(c) Scum removal.

(1) A clarifier must include scum baffles and a means for scum collection and disposal.

(A) A clarifier at a wastewater treatment facility with a design flow equal to or greater than 10,000 gallons per day must use a mechanical skimmer.

(B) A clarifier at a wastewater treatment facility with a design flow less than 10,000 gallons per day may use either mechanical skimming or hydraulic differential skimming. Hydraulic differential skimming may only be used if the scum pickup is capable of removing scum from the entire operating surface of the clarifier.

(2) Scum collected from a clarifier in a wastewater treatment facility using an activated sludge process or an aerated lagoon must be:

(A) discharged to an aeration basin or digester; or

(B) disposed of using any method that complies with Chapter 312 of this title (relating to Sludge Use, Disposal, and Transportation).

(3) If scum contains debris or foreign materials that were not removed by preliminary treatment, debris and foreign materials must be removed from the scum

before the scum is discharged to a digester or applied to the land. Otherwise, scum containing debris or other foreign materials must be sent to a landfill or a sludge-only monofill.

(4) Scum collected from a clarifier in a wastewater treatment facility not using an activated sludge process and not using an aerated lagoon must be discharged to a sludge digester or must be disposed of using any disposal method that complies with Chapter 312 of this title.

(5) Discharge of scum to a drying area that is open to the atmosphere is prohibited.

(6) Storage or holding of scum in a containment vessel that is open to the atmosphere is prohibited. Scum must be contained in a vessel that is not open to the atmosphere or must be routed to a treatment unit in the wastewater treatment facility for further processing.

(7) A pump used for pumping scum must be specifically designed to pump scum.

(d) Clarifier Effluent Weirs.

(1) A clarifier effluent weir must prevent turbulence or localized high vertical flow velocity in a clarifier.

(2) A clarifier effluent weir must be located a minimum of 6.0 inches from an outer wall or baffle, except for a clarifier effluent weir and launder that is attached to a wall. A clarifier effluent weir must prevent the short-circuiting of flow throughout the clarifier.

(3) A clarifier effluent weir must be level, and must be adjustable to allow re-leveling of the weir and to provide for minor changes to the water surface elevation in the clarifier.

(4) For a wastewater treatment facility with a design flow of less than 1.0 million gallons per day, the clarifier effluent weir loading must not exceed 20,000 gallons per day at the peak flow per linear foot of weir length.

(5) For a wastewater treatment facility with a design flow equal to or greater than 1.0 million gallons per day, the clarifier effluent weir loading must not exceed 30,000 gallons per day at the peak flow per linear foot of weir length.

(6) A center-feed circular clarifier must have effluent weirs around the entire perimeter of the clarifier.

(e) Sludge Pipes.

(1) The transfer of sludge from a clarifier to a treatment or processing unit must not negatively affect treatment efficiency of the unit that receives the sludge.

(2) A sludge pipe must be a minimum of 4.0 inches in diameter.

(3) The flow velocity in a sludge pipe must be greater than 2.0 feet per second for a wastewater treatment facility with a design flow greater than 150,000 gallons per day. For a wastewater treatment facility with a design flow of 150,000 gallons per day or less, the flow velocity in the sludge pipe must be greater than 0.5 feet per second. The executive director may consider approving lower velocities in writing for a wastewater treatment facility that uses a biological nutrient removal process, on a case-by-case basis.

(4) A sludge pipe must be accessible for cleaning.

(5) A means to remove a blockage from all sludge pipes must be provided at the wastewater treatment facility.

(f) Sludge Collection Equipment.

(1) A clarifier that is part of a wastewater treatment facility with a design flow of 10,000 gallons per day or greater must include mechanical sludge collecting equipment.

(2) A clarifier that is part of a wastewater treatment facility with a design flow of less than 10,000 gallons per day must include manual equipment designed to move settled sludge to the sludge collection pipe, unless mechanical sludge collecting equipment is provided.

(g) Side Water Depth.

(1) For a secondary clarifier, the side water depth is defined as:

(A) the water depth from the top of the cone in a cone bottom tank to the water surface; or

(B) the water depth from 2.0 feet above the bottom of a flat bottom tank to the water surface.

(2) The minimum side water depth for a clarifier with a mechanical sludge collector is:

(A) 10.0 feet if the surface area is equal to or greater than 300 square feet; and

(B) 8.0 feet if the surface area is less than 300 square feet.

(3) A clarifier with a hopper bottom must have a minimum side water depth of 8.0 feet, not including the hopper and sump. The corresponding surface area and surface loading relationships in Figure: 30 TAC §217.152(g)(3) must be used.

Figure: 30 TAC §217.152(g)(3)

Equation F.1.

$SLR = Q_{peak}/SA$

Where:

SLR = surface loading rate (gallons per square foot per day)

Q_{peak} = peak flow (gallons per day)

SA = surface area (square feet)

(4) The hopper portion of a hopper bottom clarifier must have a vertical depth of at least 4.0 feet.

(h) Restrictions on Hopper Bottom Clarifiers.

(1) A hopper bottom clarifier is prohibited for use in a wastewater treatment facility with a design flow equal to or greater than 10,000 gallons per day.

(2) Each hopper cell of a hopper bottom clarifier must have individually controlled sludge removal equipment.

(3) A hopper bottom clarifier must have a smooth wall finish.

(4) A hopper bottom clarifier must have an upper hopper slope of not less than 60 degrees from horizontal.

(i) Restrictions on Short Circuiting. The influent stilling baffle and effluent clarifier weir must prevent hydraulic short circuiting.

(j) Return Sludge Pumping Capacity.

(1) The capacity of a return sludge pumping system must be calculated based on the area of the activated sludge clarifier or clarifiers, including the stilling well area.

(2) The return sludge pumping capacity must be equal to or greater than the clarifier underflow rate in gallons per day per square foot (gpd/sf) with the largest pump out of service.

(3) A return sludge pumping system must be capable of pumping at least 200 gpd/sf but not more than 400 gpd/sf.

(4) The return sludge pumping capacity must be controlled via throttling, variable speed drives, or multiple pump operation.

§217.153. Requirements for Both Aeration Basins and Clarifiers.

(a) Construction. Construction material for aeration basins and clarifiers must be resistant to the effects of a corrosive wastewater environment.

(1) Aeration basins and clarifiers must not be buoyant when empty.

(2) Structures using a common wall must be designed to accommodate the stresses generated when one basin is full and an adjacent basin is empty.

(3) Aeration basin and clarifier walls must be watertight.

(b) Freeboard.

(1) An aeration basin must have a minimum freeboard of 18 inches at the peak flow.

(2) A clarifier must have a minimum freeboard of 12 inches at the peak flow.

(c) Redundancy and Flow Control.

(1) A wastewater treatment facility with a design flow equal to or greater than 0.4 million gallons per day must have a minimum of two aeration basins and two clarifiers. Aeration basins are exempt from this requirement if the aeration equipment, including the diffusers, is removable without taking the aeration basin out of service.

(2) Internal and interconnecting pipes must be capable of hydraulically handling the peak flow without overflow while either the largest clarifier or the largest aeration basin is out of service.

(3) Each aeration basin and clarifier must have gates or valves to allow it to be hydraulically isolated.

(4) Each aeration basin and clarifier must have a dedicated means for draining.

§217.154. Aeration Basin and Clarifier Sizing--Traditional Design.

(a) This section applies to the traditional approach for sizing an aeration basin and clarifier, and is based on empirically-derived design values that have been used historically as standard engineering practice.

(b) Aeration Basin Sizing.

(1) An aeration system must be designed to maintain a minimum dissolved oxygen concentration of 2.0 milligrams per liter (mg/l) throughout the aeration basin at the maximum diurnal organic loading rate determined in §217.32(a)(3) and Figure: 30 TAC §217.32(a)(3) of this title (relating to Organic Loading and Flows for New Wastewater Treatment Facilities). The executive director may consider alternative dissolved oxygen specifications for designs that include biological nutrient removal or tapered aeration for energy conservation, which must be approved in writing.

(2) Based on the calculated organic load, the aeration basin volume must be designed to ensure that the organic loading on the aeration basin does not exceed the organic loading rates in the following table:

Figure: 30 TAC §217.154(b)(2)

Table F.1. - Design Organic Loading Rates for Sizing Aeration Basins Based on Traditional Design Methods

Process	**Applicable Permit Effluent Sets** *Concentration milligrams per liter (mg/l)*			**Maximum Organic Loading Rate** *Pounds BOD_5/day/1,000 cubic feet (lbs/day/1,000cf)*
	Five-day Biochemical Oxygen Demand (BOD_5)	**Total Suspended Solids (TSS)**	**Ammonia Nitrogen**	
Conventional activated sludge process without nitrification	10	15	NA	45
	20	20	NA	
Conventional activated sludge process with nitrification when reactor temperatures exceed 15˚ C	10	15	3, 2, or 1	35
Conventional	10	15	3, 2, or 1	25

activated sludge process with nitrification when reactor temperatures are 13˚ to 15˚ C				
Conventional activated sludge process with nitrification when reactor temperatures are 10˚ to 12˚ C	10	15	3, 2, or 1	20
Extended aeration basins including oxidation ditches (mean cell residence time over 20 days)	10	15	3, 2, or 1	15

(3) When identifying the aeration basin temperature for the process design in Table F.1. in Figure: 30 TAC §217.154(b)(2), the owner must use the average of the lowest consecutive seven-day mean aeration basin temperature from a wastewater treatment facility with similar characteristics. For purposes of this subsection, a similar wastewater treatment facility:

(A) is located within 50 miles of the wastewater treatment facility where the planned aeration basin or system will be installed;

(B) uses the same placement of the aeration basin (in-ground or above-ground); and

(C) has any other characteristics required by the executive director in writing.

(c) Clarifier Sizing.

(1) The following table establishes the maximum surface loading rates and the minimum detention times that must be used to determine the size of a clarifier:

Figure: 30 TAC §217.154(c)(1)

Table F.2. - Maximum Clarifier Weir Overflow Rates Based Upon Traditional Design Methods

<table>
<tr><th colspan="3">Applicable Permit Effluent Sets concentration milligrams per liter (mg/l)</th><th rowspan="2">Aeration Basin Organic Loading (five-day biochemical oxygen demand, from Table F.1.</th><th rowspan="2">Process - Treatment Level</th><th rowspan="2">Maximum Surface Loading Rate at Two-Hour Peak Flow (gallons/day/ square foot)</th><th rowspan="2">Minimum Detention Time at Two-Hour Peak Flow (hours)</th></tr>
<tr><th>BOD_5</th><th>TSS</th><th>NH_3-N</th></tr>
<tr><td>20</td><td>20</td><td></td><td rowspan="2">45</td><td rowspan="2">Fixed film - secondary or enhanced secondary</td><td rowspan="2">1,200</td><td rowspan="2">1.8</td></tr>
<tr><td>10</td><td>15</td><td></td></tr>
<tr><td>20</td><td>20</td><td></td><td rowspan="3">45, 35, 25 or 20</td><td rowspan="3">Activated sludge - secondary, enhanced secondary, or secondary with nitrification</td><td rowspan="3">1,200</td><td rowspan="3">1.8</td></tr>
<tr><td>10</td><td>15</td><td></td></tr>
<tr><td>10</td><td>15</td><td>3</td></tr>
<tr><td>20</td><td>20</td><td></td><td>15</td><td>Extended air - secondary</td><td>900</td><td>2.0</td></tr>
<tr><td>10</td><td>15</td><td>3</td><td>15</td><td>Extended aeration - enhanced secondary</td><td>800</td><td>2.2</td></tr>
</table>

(2) A clarifier must meet both the detention time and weir overflow rate criteria in Table F.2. in Figure: 30 TAC §217.154(c)(1).

(A) When calculating weir overflow rates for a clarifier, return activated sludge flow must not be used in the calculation of the maximum weir overflow rate, in compliance with Table F.2. in Figure: 30 TAC §217.154(c)(1).

(B) When calculating the weir overflow rate for a clarifier, the surface area of the stilling well may be included as part of the clarifier surface area.

§217.155. Aeration Equipment Sizing.

(a) Oxygen Requirements (O_2 R) of Wastewater.

(1) An aeration system must be designed to provide a minimum dissolved oxygen concentration in the aeration basin of 2.0 milligrams per liter (mg/l).

(2) Mechanical and diffused aeration systems must supply the O_2 R calculated by Equation F.2. in Figure: 30 TAC §217.155(a)(3), or use the recommended values presented in Table F.3. in Figure: 30 TAC §217.155(a)(3), whichever is greater.

(3) The O_2 R values in Table F.3. in Figure: 30 TAC §217.155(a)(3) use concentrations of 200 mg/l five-day biochemical oxygen demand (BOD_5) and 45 mg/l ammonia-nitrogen (NH3-N) in Equation F.2. in Figure: 30 TAC §217.155(a)(3):

Figure: 30 TAC §217.155(a)(3)

Equation F.2.

$$O_2R = \frac{1.2(BOD_5) + 4.3(NH_3 - N)}{BOD_5}$$

Where:
O_2R = Oxygen requirement, pound (lb) O_2/lb five-day biochemical oxygen demand (BOD_5)
BOD_5 = BOD_5 concentration, milligrams per liter (mg/L)
NH_3-N = Ammonia nitrogen, mg/L

Table F.3. - Minimum O_2R for Lower BOD_5 Loadings

Process	O_2R, pounds (lbs) O_2/lb BOD_5
Conventional Activated Sludge Systems that are not Intended to Nitrify	1.2
Conventional Activated Sludge Systems that are Intended to Nitrify and Extended Aeration Systems (including all Oxidation Ditch Treatment Systems)	2.2

(b) Diffused Aeration System. An airflow design must be based on either paragraph (1) or (2) of this subsection.

(1) Design Airflow Requirements - Default Values. A diffused aeration system may use Table F.4. in Figure: 30 TAC §217.155(b)(1) to determine the airflow for sizing aeration system components:

Figure: 30 TAC §217.155(b)(1)

Table F.4. - Minimum Airflow Requirements for Diffused Air

Process	**Airflow/Five-day biochemical oxygen demand (BOD_5) load** *standard cubic feet/day/pound*
Conventional activated sludge systems that are not intended to nitrify	1,800*
Extended aeration systems and all other activated sludge systems that are intended to nitrify, including all oxidation ditch	3,200*

treatment systems	
*These values were calculated using Equation F.3. in Figure: 30 TAC §217.155(b)(2)(A)(iv) with the following assumptions: a transfer efficiency of 4.0% in wastewater for all diffused air activated sludge processes; a diffuser submergence of 12 feet; a wastewater temperature of 20˚C; and the oxygen requirements in Figure: 30 TAC §217.155(a)(3), Table F.3.	

(2) Design Airflow Requirements - Equipment and Site Specific Values. A diffused aeration system may be based on calculations of the airflow requirements for the diffused aeration equipment in accordance with subparagraphs (A) - (D) of this paragraph.

(A) Determine Clean Water Oxygen Transfer Efficiency.

(i) A diffused aeration system design may be based on a clean water oxygen transfer efficiency greater than 4%, only if the clean water oxygen transfer efficiency is supported by full scale diffuser performance data. Full scale performance data must be developed by an accredited testing laboratory or a licensed professional engineer. Data developed by a professional engineer must be sealed by the engineer.

(ii) A testing laboratory or licensed engineer shall use the oxygen transfer testing methodology described in the most current version of the American Society of Civil Engineers (ASCE) publication, A Standard for the Measurement of Oxygen Transfer in Clean Water.

(iii) A diffused aeration system with a clean water transfer efficiency greater than 18% for a coarse bubble system or greater than 26% for a fine bubble system is considered an innovative technology and is subject to §217.7(b)(2) of this title (relating to Types of Plans and Specifications Approvals).

(iv) A design for clean water transfer efficiencies obtained at temperatures other than 20 degrees Celsius must be adjusted for a diffused aeration system to reflect the approximate transfer efficiencies and air requirements under field conditions by using the following equation:

Figure: 30 TAC §217.155(b)(2)(A)(iv)

<u>**Equation F.3.**</u>

$$FTE = (T_e) \times \left(\frac{WOTE}{CWOTE}\right) \times 1.024^{T-20} \times \left(\frac{C_f}{C_t}\right)$$

<u>Where:</u>
T_e = Test Efficiency
FTE = Field Transfer Efficiency (decimal)
WOTE = Wastewater Oxygen Transfer Efficiency (decimal)
CWOTE = Clean Water Oxygen Transfer Efficiency (decimal)
T = Temperature (degrees C)
C_f = Oxygen Saturation in Field (Includes temperature, dissolved solids, pressure, etc.)
C_t = Oxygen Saturation in Test Conditions

(B) Determining Wastewater Oxygen Transfer Efficiency (WOTE).

(i) The WOTE must be determined from clean water test data by multiplying the clean water transfer efficiency by 0.65 for a coarse bubble diffuser or by multiplying the clean water transfer efficiency by 0.45 for a fine bubble diffuser.

(ii) The executive director may require additional testing and data to justify actual WOTE for a wastewater treatment facility treating wastewater containing greater than 10% industrial wastes.

(C) Determining Required Airflow (RAF). The RAF must be calculated using the following equation to determine the size needed for a diffuser submergence of

12.0 feet. If the diffuser submergence is other than 12.0 feet, a diffused aeration system must correct the RAF, as detailed in subparagraph (D) of this paragraph.

Figure: 30 TAC §217.155(b)(2)(C)

Equation F.4.

$$RAF = \frac{(PPD\ BOD_5) \times (O_2 / lb\ BOD_5)}{WOTE \times 0.23 \times 0.075 \times 1440}$$

Where:
RAF = Required Airflow Rate (standard cubic feet per minute (SCFM))
PPD BOD_5 = Influent Organic Load in Pounds per Day of five-day biochemical oxygen demand
0.23 = lb 0_2/lb air @ 20˚ C
1440 = minutes/day
0.075 = lb air/cubic foot (cf)
WOTE = Wastewater Oxygen Transfer Efficiency (decimal)
If the design inlet temperature is above 24˚ C, the specific weight of air must be adjusted to the specific weight at the intake temperature.

(D) Corrections to RAF based on varying diffuser submergence depths. The engineer shall provide the manufacturer's laboratory testing data if the diffuser submergence depth in the design is the same as the diffuser submergence depth in the manufacturer's testing. The engineer shall apply a correction factor from Table F.5. in Figure: 30 TAC §217.155(b)(2)(D) to the required airflow rate calculated using Equation F.4. in Figure: 30 TAC §217.155(b)(2)(C) if the manufacturer's laboratory testing data is not available for the design diffuser submergence depth. Linear interpolation is allowed for diffuser submergence depths not shown in Table F.5. in Figure: 30 TAC §217.155(b)(2)(D).

Figure: 30 TAC §217.155(b)(2)(D)

Table F.5. - Diffuser Submergence Correction Factors	
Diffuser Submergence Depth *(feet)*	**Airflow Rate Correction Factor**
8	1.82
10	1.56
12	1.00
15	0.91
18	0.73
20	0.64

(3) Mixing Requirements for Diffused Air. The air requirements for mixing must be calculated using an airflow rate:

(A) from Design of Municipal Wastewater Treatment Plants, Fifth Edition, Chapter 11, a joint publication of the ASCE and the Water Environment Federation, for mixing requirements; or

(B) greater than or equal to 20 standard cubic feet per minute (scfm) per 1,000 cubic feet for a coarse bubble diffuser and greater than or equal to 0.12 scfm per square foot for a fine bubble diffuser.

(4) Blowers and Air Compressors.

(A) A blower and an air compressor system must provide the required design airflow rate for biological treatment and mixing, based on paragraphs (1) - (3) of this subsection, and the air requirements of all other supplemental units where air must be supplied.

(B) The engineering report must include blower and air compressor calculations that show the maximum air requirements for the temperature range where the wastewater treatment facility is located, including both summer and winter conditions, and the impact of elevation on the air supply.

(C) A diffused aeration system must have multiple compressors arranged to provide an adjustable air supply to meet the variable organic load on the wastewater treatment facility.

(D) The air compressors must be capable of handling the maximum design air requirements with the largest single air compressor out of service.

(E) A blower unit and a compressor unit must restart automatically after a power outage, or have a telemetry system or an auto-dialer with battery backup to notify an operator of any outage.

(F) The design of a blower and air compressor system must specify blowers and air compressors with sufficient capacity to handle air intake temperatures that may exceed 100 degrees Fahrenheit (38 degrees Celsius), and pressures that may be less than standard (14.7 pounds per square inch absolute).

(G) The design of a blower and air compressor system must specify the capacity of the motor drive necessary to handle air intake temperatures that may be 20 degrees Fahrenheit (-7 degrees Celsius) or less.

(H) A blower must include a governor or other means to regulate airflow.

(5) Diffuser Systems - Additional Requirements.

(A) Diffuser Submergence.

(i) For a new wastewater treatment facility, the submergence depth for any diffuser must meet the minimum depths in the following table:

Figure: 30 TAC §217.155(b)(5)(A)(i)

Table F.6. - Minimum Diffuser Submergence Depth	
Design Flow (*mgd)*	**Minimum Submergence Depth** *(feet)*
<0.01	8.0
0.01 to 0.10	9.0
>0.10	10.0

(ii) For an alteration or expansion of an existing wastewater treatment facility, the diffuser submergence depth may vary from the values in Table F.6. in Figure: 30 TAC §217.155(b)(5)(A)(i) to match existing air pressure, delivery rate, and hydraulic requirements.

(iii) The submerged depth for a diffuser must be at least 7.0 feet. A wastewater treatment facility with a design flow of less than 5,000 gallons per day may have a diffuser submergence depth of less than 7.0 feet, but only if justified by the engineer and approved in writing by the executive director.

(B) Grit Removal. A wastewater treatment facility that uses diffusers and has wastewater with concentrations of grit that would interfere with the operation of a diffuser must either include a grit removal unit upstream of an aeration process, or include multiple aeration basins so that one basin may be taken out of service to allow for grit removal.

(C) Aeration System Pipes.

(i) Each diffuser header must include an open/close or throttling type control valve that can withstand the heat of compressed air.

(ii) A diffuser header must be able to withstand temperatures up to 250 degrees Fahrenheit.

(iii) The capacity of an air diffuser system, including pipes and diffusers, must equal 150% of design air requirements.

(iv) The design of an aeration system must minimize head loss. The engineering report must include a hydraulic analysis of the entire air pipe system that quantifies head loss through the pipe system and details the distribution of air from the blowers to the diffusers.

(v) An aeration system may use non-metallic pipes only in the aeration basin, but the pipes must be a minimum of 4.0 feet below the average water surface elevation in the aeration basin.

(c) Mechanical Aeration Systems.

(1) Required Airflow - Equipment and Site Specific Values. The airflow requirements for a mechanical aeration system must be calculated in accordance with subparagraphs (A) and (B) of this paragraph.

(A) Clean Water Oxygen Transfer Efficiency.

(i) The engineering report must include the clean water oxygen transfer efficiency rate for the mechanical equipment.

(ii) The clean water oxygen transfer efficiency must not exceed 2.0 pounds of oxygen per horsepower-hour unless justified by full scale performance data. Full scale performance data must be developed by an accredited testing laboratory or a licensed professional engineer. Data developed by a professional engineer must be sealed by the engineer. Full scale performance tests must follow the oxygen transfer testing methodology described in the most current version of the ASCE publication, A Standard for the Measurement of Oxygen Transfer in Clean Water.

(iii) A technology with a proposed clean water oxygen transfer efficiency in excess of 2.0 pounds of oxygen per horsepower-hour is innovative technology and subject to the requirements of §217.7(b)(2) of this title (relating to Types of Plans and Specifications Approvals).

(B) Wastewater Oxygen Transfer Efficiency.

(i) The engineering report must include the actual wastewater oxygen transfer efficiency and data to justify the actual wastewater oxygen transfer efficiency.

(ii) If a wastewater treatment facility will receive more than 10% industrial wastewater by volume, all mechanical aeration equipment must be sized based on a wastewater oxygen transfer efficiency of no more than 0.65 times the clean water oxygen transfer efficiency.

(2) Mixing Requirements.

(A) A mechanical aeration device must provide mixing to prevent mixed liquor suspended solids (MLSS) deposits under any flow condition.

(B) A mechanical aeration device must be capable of re-suspending the MLSS after a shutdown period.

(C) Mechanical aeration devices with a channel or basin layout must have a minimum of 100 horsepower per million gallons of aeration basin volume or 0.75 horsepower per thousand cubic feet of aeration basin volume.

(3) Mechanical Components.

(A) Process Reliability.

(i) Each aeration basin must include a minimum of two mechanical aeration devices.

(ii) A mechanical aeration device must meet the maximum design requirements for oxygen transfer with the largest single unit out of service.

(iii) A mechanical aeration device must either automatically restart after a power outage, or have a telemetry system or an auto-dialer with battery backup to notify an operator of any outage.

(B) Operation and maintenance.

(i) A mechanical aeration device must have two-speed or variable-speed drive units, unless another means of varying the output is provided.

(ii) To vary the output, a mechanical aeration device may use single-speed drive units with timer-controlled operation if the device also includes an independent means of mixing.

(iii) A wastewater treatment facility must be designed such that an operator is able to perform routine maintenance on the aeration equipment without coming into contact with wastewater.

(iv) Each bearing, drive motor, or gear reducer must be accessible to an operator for maintenance and must be equipped with a splash prevention device. A splash prevention device must be designed to protect the operator from contact with wastewater and to prevent wastewater from escaping the basin.

(v) Each gear reducer must have a drainage system to prevent operator contact with mixed liquor.

§217.156. Sequencing Batch Reactors.

(a) System Sizing and Reliability.

(1) A sequencing batch reactor (SBR) must meet the reliability requirements in §217.155(b) and (c)(3) of this title (relating to Aeration Equipment Sizing), and power source reliability requirements in §217.36 of this title (relating to Emergency Power Requirements).

(2) An SBR must have a minimum decantable volume that is sufficient to pass the [design] permitted peak flow, and must be capable of meeting permitted effluent limits, with the largest basin out of service.

(3) A two-basin wastewater treatment facility without removable aeration devices is required to have aerated storage of mixed liquor separate from the SBR tank(s).

(4) An SBR with a fixed level decanter must have more than two basins and additional decantable storage volume.

(5) An equalization basin is required if an SBR has fixed decant equipment and decant volumes that do not accommodate the peak flow.

(6) Organic loadings must conform to the values in Table F.1. in Figure: 30 TAC §217.154(b)(2) of this title (relating to Aeration Basin and Clarifier Sizing--Traditional Design). Organic loadings must be below 35 pounds of five-day biochemical oxygen demand per 1,000 cubic feet of tank volume.

(7) The reactor mixed liquor suspended solids (MLSS) level at the normal operating level must range from at least 3,000 milligrams per liter (mg/l) to not more than 5,000 mg/l.

(8) The minimum depth of the MLSS during a react phase is 9.0 feet.

(9) The minimum side water depth of an SBR tank is 12 feet.

(10) An SBR must include sludge digestion pursuant to the requirements in Subchapter J of this chapter (relating to Sludge Processing).

(b) Decanter Design.

(1) A decanter must control the velocity at an inlet port or at the edge of submerged weirs to prevent vortexing, disturbance of the settled sludge, and entry of floating materials.

(2) The entrance velocity to a decanter must not exceed 1.0 foot per second.

(3) A decanter must draw effluent from below the water surface and include a device that excludes scum.

(4) A decanter must maintain a zone of separation between the settled sludge and the decanter of no less than 12 inches.

(5) A decanter must prevent solids from entering the decanter during a react cycle by using one the following methods:

(A) recycling treated effluent to wash out solids trapped in a decanter;

(B) mechanically closing a decanter when it is not in use; or

(C) filling a decanter with air except during a decant period.

(6) The performance of a decanter and related pipes and valves must not be affected by ambient temperatures below 32 degrees Fahrenheit.

(7) A fixed decanter is prohibited in a basin where simultaneous fill and decant may occur.

(8) For any system of tanks that is fed sequentially, the size of the decant system must accommodate the design flow with a constant cycle time with the largest tank out of service.

(9) An SBR system utilizing more than two basins must allow the decanting of at least two tanks simultaneously.

(10) If units downstream of an SBR are not capable of accepting the peak flow rate of the decanting cycle, flow equalization must be provided between the decanter and the downstream units.

(c) SBR Tank Details.

(1) An SBR requires multiple tanks.

(2) An SBR with two tanks or an SBR system operating with a continuous feed during settling and decanting phases must include influent baffling and physical separation from the decanter.

(3) An elongated tank must be used for an SBR system if influent baffling is required.

(4) An SBR tank must have a minimum freeboard of 18 inches at the maximum liquid level.

(5) An SBR tank must not be buoyant when empty.

(6) Structures using a common wall must be designed to accommodate the stresses generated when one basin is full and an adjacent basin is empty.

(7) Each SBR wall must be watertight.

(8) A sump must be provided in any basin with a flat bottom.

(9) An SBR system must have a dedicated means of transferring sludge between aeration basins.

(10) An SBR system must include a means of scum removal in each aeration basin.

(11) Each SBR tank must include a dewatering system and an emergency overflow to another aeration tank or a storage tank.

(12) At a wastewater treatment facility that is not staffed 24-hours per day, a manually operated SBR tank must include a high-level alarm that notifies wastewater treatment facility staff in accordance with §217.161 of this title (relating to Electrical and Instrumentation Systems). Any communication system must be able to notify staff on either loss of power or communication.

(13) A design must specify the means and frequency for removal of grit and other debris from the SBR tanks.

(14) All equipment must be accessible for inspection, maintenance, and operation. Walkways shall be provided to allow inspection, maintenance, and process control sampling and to allow access to instrumentation, mechanical equipment, and electrical equipment.

(15) An SBR may use fine screens pursuant to §217.122 of this title (relating to Fine Screens).

(16) An SBR preceded by a primary clarifier may use a comminutor.

(17) An SBR must have a sufficient number of tanks to operate at [design] permitted peak flow with the largest tank out of service.

(d) Aeration and Mixing Equipment.

(1) In addition to the requirements of §217.155 of this title, aeration equipment must handle the cyclical operation in an SBR.

(2) The aeration and mixing equipment must not interfere with settling.

(3) A dissolved oxygen concentration of 2.0 mg/l must be maintained in a tank during the fill cycle; unless the system is design for biological nutrient removal.

(4) The design must specify the blower discharge pressure at the maximum water depth.

(5) An SBR used for biological nutrient removal or reduction must meet the design requirements of §217.163 of this title (relating to Advanced Nutrient Removal).

(6) The design of an SBR must allow for the removal of air diffusers or mechanical aeration devices without dewatering the tank.

(7) The oxygen concentration range used for sizing aeration systems for treatment zones must be:

(i) not more than 0.5 mg/l for anoxic basins;

(ii) at least 1.5 mg/l but not more than 3.0 mg/l for aerobic basins; and

(iii) at least 2.0 mg/l but not more than 8.0 mg/l for membrane basins

(e) Control Systems.

(1) The [motor] control center must include programmable logic controllers (PLC) that are able to operate with limited operator adjustment and be programmed to meet the effluent limitations in the wastewater permit at the design loadings. An SBR must have the ability to run in full manual mode. If there is failure of any control system notification should be sent to the operator.

(2) A hard-wired backup means of operating the SBR is required.

(3) The PLC must include battery backup. A duplicate set of all circuit boards must be kept at the wastewater treatment facility.

(4) Adequate controls for the separate operation of each tank must be provided.

(5) A tank level system must include floats or pressure transducers.

(A) A float system must be protected from prevailing winds and freezing.

(B) A bubbler system in a tank level system is prohibited.

(6) The control panel switches must include the following switches:

(A) pumps - hand/off/automatic;

(B) valves - open/closed/automatic;

(C) blowers or aerators - hand/off/automatic; and

(D) selector switch for tank(s) - in operation/standby.

(7) The control panel visual displays must include:

(A) a mimic diagram of the process that shows the status and position of all pumps, valves, blowers, aerators, and mixers;

(B) process cycle and time remaining;

(C) instantaneous and totalized influent flow to the wastewater treatment facility and effluent flow of the final discharge;

(D) tank level gauges or levels;

(E) sludge pumping rate and duration; and

(F) airflow rate and totalizer.

(8) The annunciator panel must include the following alarm condition indicators:

(A) high and low water levels in each tank;

(B) failure of all automatically operated valves;

(C) decanter failure;

(D) blowers, if used - low pressure, high temperature, and failure;

(E) mechanical aerator, if used - high temperature and failure;

(F) pump - high pressure and failure; and

(G) mixers, if used - failure.

TCEQ is still reviewing technology standards and working on drafting the flux rates and other issued for MBRs. Comments are welcome

§217.157. Membrane Bioreactor Systems.

(a) Applicability.

(1) This section contains criteria for low-pressure, vacuum, and gravity ultrafiltration or microfiltration membrane bioreactors (MBRs).

(2) Other types of MBRs are considered innovative technology and are subject to the requirements of §217.7(b)(2) of this title (relating to Types of Plans and Specifications Approvals).

(b) Definitions.

(1) Flat plate system--A membrane bioreactor that arranges membranes into rectangular cartridges with a porous backing material that provides structural support between two membranes.

(2) Gross flux rate--The volume of water that passes through a membrane, measured in gallons per day per square-foot of membrane area at a standard temperature of 20 degrees Centigrade.

(3) Hollow fiber system--A membrane bioreactor composed of bundles of very fine membrane fibers, approximately 0.5 - 2.0 millimeters in diameter, held in place at the ends with hardened plastic potting material, and supported on stainless steel

frames or rack assemblies. The outer surface of each fiber is exposed to the mixed liquor with filtrate flow from outside to inside through membrane pores.

(4) Net flux rate--The gross flux rate adjusted for production lost during backwash, cleaning, and relaxation.

(5) Transmembrane pressure--The difference between the average pressure on the feed side of a membrane and the average pressure on the permeate side of a membrane.

(6) Tubular system--A system in which sludge is pumped from an aeration basin to a pressure driven membrane system outside of a bioreactor where the suspended solids are retained and recycled back into the bioreactor while the effluent passes through a membrane.

(c) Performance Standards.

(1) MBR performance standards for conventional pollutants and nutrients are shown in the following table:

Figure: 30 TAC §217.157(c)(1)

Table F.7. Performance Standards for Conventional Pollutants and Nutrients

Parameter	Units	Expected Value
Five-day Biochemical Oxygen Demand	*milligrams per liter (mg/l)*	5

TSS	*mg/l*	1
Ammonia	*mg/l as N*	1
Total Nitrogen (with only preanoxic zone)	*mg/l*	10
Total Nitrogen (with preanoxic and postanoxic zones)	*mg/l*	3
Total Phosphorous (with chemical addition)	*mg/l*	0.2
Total Phosphorous (with bio-P removal)	*mg/l*	0.5
Turbidity	*nephelometric turbidity units*	0.2
Bacteria	*log removal*	≤ 6 log (99.9999%)
Viruses	*log removal*	≤ 3 log (99.9%)

(2) The executive director may require an owner to submit a pilot study report or data from a similar wastewater treatment facility if a wastewater treatment facility is designed to achieve higher quality effluent than the performance standards listed in Table F.7. in Figure: 30 TAC §217.157(c)(1). A similar wastewater treatment facility must have similar characteristics including:

(A) climate region;

(B) peak flows;

(C) customer base, including sources and percent contribution; and

(D) other characteristics required by the executive director.

(d) Wastewater Treatment Facility Design.

(1) Pretreatment.

(A) Each MBR system must have fine screening to prevent damage from abrasive particles or fibrous, stringy material.

(i) Fine screens must be rotary drum or traveling band screens with either perforated plates or wire mesh.

(ii) Fine screens for hollow fiber or tubular systems must have an opening size of 0.5 - 2.0 millimeter (mm).

(iii) Fine screens for flat plate systems must have an opening size of 2.0 - 3.0 mm.

(iv) Bypass of a fine screen must be prevented by use of a duplicate fine screen, emergency overflow to a wet well, or an alternative method that has been approved in writing by the executive director.

(v) A fine screen must be designed to prevent bypass at the peak flow.

(vi) Coarse screens may be used ahead of fine screens to reduce the complications of fine screening.

(B) The economic feasibility of primary sedimentation must be evaluated for facilities designed for an average daily flow of 5.0 million gallons per day or more. The economic feasibility evaluation must be included in the engineering report.

(C) Fat, oil, and grease removal is required if the levels of fat, oil, and grease in the influent may cause damage to the membranes. The specific detrimental concentration must be determined by the equipment manufacturer. Influent concentrations of fat, oil, and grease equal to or more than 100 milligrams per liter (mg/l) must have fat, oil, and grease removal. Is this still an appropriate concentration???

(D) The necessity of grit removal must be evaluated for a wastewater treatment facility that has a collection system with excessive inflow and infiltration. Excessive grit accumulation is characterized by grit accumulation in any treatment unit following the headworks. An evaluation must be included in the engineering report.

(2) Biological Treatment.

(A) The reactor volume must be determined using rate equations for substrate utilization and biomass growth according to §217.154 of this title (relating to Aeration Basin and Clarifier Sizing--Traditional Design), or another method approved by the executive director in writing. Combined aeration-MBR basins must discount the volume of the MBR banks/units to determine compliance with §217.154 requirements for aeration basin sizing.

(B) The design sludge retention time (SRT) for an MBR must be at least 10 days, but not more than 25 days.

(C) The design operational range of mixed liquor suspended solids (MLSS) concentration must be:

(i) at least 4,000 mg/l but not more than 10,000 mg/l in the bioreactor; and

(ii) at least 4,000 mg/l but not more than 14,000 mg/l in the membrane tank.

(D) An MBR system designed for an SRT or MLSS outside the ranges in subparagraph (C) of this paragraph requires a pilot study in compliance with paragraph (8) of this subsection or data from a similar wastewater treatment facility that demonstrates that the design parameters are sustainable and can achieve the expected performance to the executive director's satisfaction.

(3) Aeration.

(A) An aeration system in an MBR must be capable of maintaining dissolved oxygen levels as listed in subparagraph (C) of this paragraph.

(B) An aeration system in an MBR must compensate for low oxygen transfer efficiency associated with the maximum MLSS concentrations established in paragraph (2)(C) of this subsection. The alpha value used to determine design oxygen transfer efficiency must be 0.5 or lower.

(C) The oxygen concentration range used for sizing aeration systems for treatment zones must be:

(i) not more than 0.5 mg/l for anoxic basins;

(ii) at least 1.5 mg/l but not more than 3.0 mg/l for aerobic basins; and

(iii) at least 2.0 mg/l but not more than 8.0 mg/l for membrane basins.

(D) An MBR must include dissolved oxygen monitoring and an alarm to notify an operator if dissolved oxygen levels are outside of the design operating range, or if there is a rapid decrease in dissolved oxygen. Alarm systems must comply with §217.161 of this title (relating to Electrical and Instrumentation Systems).

(4) Recycle Rates. Facilities without advanced nutrient removal must be designed with recycle rates sufficient to sustain the design mixed liquor concentrations (typically from 200% to 400% of the wastewater treatment facility's influent flow).

(5) Nutrient Removal.

(A) A system designed for advanced nutrient removal must include an isolated tank or baffled zone to separate anoxic, anaerobic, and aerobic treatment zones.

(B) The engineer shall submit calculations to support the sizing of the reactor volumes.

(C) If recycled activated sludge is returned to an anoxic or anaerobic basin, a wastewater treatment facility designed for total nitrogen or advanced nutrient removal must contain a deoxygenation basin, a larger anoxic basin, or another method of decreasing dissolved oxygen concentration approved in writing by the executive director.

(D) An advanced nutrient removal system must be designed with recycle rates sufficient to sustain the designed mixed liquor concentrations in both the aeration, anoxic, and anaerobic basins (sufficient recycle rates are typically 600% or more of the influent flow).

(6) Use of Membranes.

(A) Use of a membrane system other than a hollow fiber system, tubular system, or a flat plate system is considered an innovative technology and is subject to §217.7(b)(2) of this title.

(B) The engineering report must provide justification for the use of a membrane material other than one of the following:

(i) polyethersulfone (PES);

(ii) polyvinylidene fluoride (PVDF);

(iii) polypropylene (PP);

(iv) polyethylene (PE);

(v) polyvinylpyrrolidone (PVP); or

(vi) chlorinated polyethylene (CPE).

(vii) ceramic

(C) The nominal pore size used in an MBR for microfiltration membranes must be at least 0.10 micrometers (microns) but not more than 0.4 microns.

(D) The nominal pore size used in an MBR for ultrafiltration must be at least 0.02 microns but not more than 0.10 microns.

(E) Any chemical used for cleaning must not adversely affect the membrane material.

(7) Non-Ceramic Membrane Design Parameters.

(A) MBRs must be designed for:

(i) an average daily net flux rate equal to or less than 15 gallons per day per square-foot of membrane area;

(ii) a peak daily net flux rate equal to or less than 1.25 times the average daily net flux rate; and

(iii) a two-hour peak net flux rate equal to or less than 1.5 times the average daily net flux rate.

(B) The executive director may approve larger net flux rates if the rates are substantiated to the executive director's satisfaction with a pilot study or data from a similar wastewater treatment facility.

(C) An MBR system with a peak flow rate that is greater than 2.5 times the average daily flow must use an equalization basin, off-line storage, or reserve membrane capacity to accommodate the higher peak flow.

(D) Hollow Fiber Transmembrane Pressure (TMP).

(i) The operational TMP of a hollow fiber MBR system must be at least 2.0 pounds per square inch (psi) but not more than 10.0 psi.

(ii) The TMP of a hollow fiber MBR system must not exceed 12.0 psi.

[(E) Flat Plate TMP.

(i) The operational TMP of a flat plate MBR system must be at least 0.3 psi but not more than 3.0 psi.

(ii) The TMP of a flat plate MBR system must never exceed 4.5 psi.]

(E) [(F)] Tubular, Out of Basin TMP.

(i) The operational TMP of a tubular, out of basin MBR system must be at least 0.5 psi but not more than 5.0 psi.

(ii) The TMP of a tubular, out of basin MBR system must never exceed 10.0 psi.

(8) Ceramic Membrane Design Parameters

(A) MBRs must be designed for:

(i) an average daily net flux rate equal to or less than 15 gallons per day per square-foot of membrane area;

(ii) a peak daily net flux rate equal to or less than 1.25 times the average daily net flux rate; and

(iii) a two-hour peak net flux rate equal to or less than 1.5 times the average daily net flux rate.

(B) The executive director may approve larger net flux rates if the rates are substantiated to the executive director's satisfaction with a pilot study or data from a similar wastewater treatment facility

(C) An MBR system with a peak flow rate that is greater than 2.5 times the average daily flow must use an equalization basin, off-line storage, or reserve membrane capacity to accommodate the higher peak flow.

(D) Flat Plate TMP.

(i) The operational TMP of a flat plate MBR system must be at least 0.3 psi but not more than 3.0 psi.

(ii) The TMP of a flat plate MBR system must never exceed 4.5 psi.

(9) [(8)] Supporting Data. An owner must provide pilot study reports or data from a similar wastewater treatment facility for a wastewater treatment facility that is either:

(A) required to meet stricter standards than in Table F.7. in Figure: 30 TAC §217.157(c)(1); or

(B) designed to operate outside normal operating parameters defined within this section.

(i) A pilot study must be conducted for at least 30 days after the initial start-up and acclimation period.

(ii) A pilot study must be designed to evaluate the membrane performance under actual operational conditions, including flow variations and influent wastewater characteristics.

(iii) The treatment and pretreatment processes evaluated in a pilot study or similar wastewater treatment facility must be equivalent to the processes that will be used in the wastewater treatment facility.

(iv) The results of the pilot study must include the following recommendations:

(I) net flux rates for design flow and peak flow;

(II) average and maximum transmembrane pressure;

(III) cleaning and backwash intervals;

(IV) expected percent chemical recovery after chemical cleaning;

(V) dissolved oxygen concentrations for reactors and membrane basins;

(VI) MLSS concentrations for reactors and membrane basins;

(VII) SRTs for reactors and membrane basins; and

(VIII) expected effluent concentrations of conventional pollutants and nutrients, including the pollutants and nutrients that will be limited or monitored in the wastewater treatment facility's wastewater permit.

(10) [(9)] Redundancy.

(A) A wastewater treatment facility must be able to operate at normal operating parameters and conditions for design flow with the largest MBR unit or train out of service.

(B) Acceptable methods of providing redundancy are additional treatment trains, additional treatment units, or storage. The engineering report must include calculations that demonstrate adequate redundancy within the wastewater treatment facility.

(11) [(10)] Other Components.

(A) Mixers.

(i) Unaerated (deoxygenation, pre/post anoxic, and anaerobic) zones must have a submersible mixing system, or an alternative mixing system that has been approved in writing by the executive director.

(ii) Coarse bubble air diffusers may be used for mixing in a pre-anoxic tank.

(B) Scum and Foam Handling. An MBR must control scum and foam so that scum or foam does not interfere with treatment and must prevent unauthorized discharge of scum or foam from a treatment unit.

(C) Cranes and Hoists. A crane, hoist, or other process or mechanism approved in writing by the executive director must be provided for periodic cleaning and maintenance of the membranes.

(12) [(11)] Disinfection.

(A) An owner may request and the executive director may approve, in writing, decreased ultraviolet light or chlorine dosing requirements for MBR effluent.

(B) the design for ultraviolet light disinfection for MBR effluent that is based on greater than 75% transmissivity must be justified in the engineering report.

(e) MBR operation.

(1) Membrane cleaning. The following methods may be used to clean membranes:

(A) air scouring of at least 0.01 standard cubic feet per minute of air per square foot of membrane area, but not more than 0.04 standard cubic feet per minute of air per square foot of membrane area;

(B) a mixture of air scouring as described in subparagraph (A) of this paragraph and mixed liquor jet feed;

(C) back-flushing;

(D) relaxation; or

(E) chemical cleaning.

(2) Operational Control Parameters.

(A) In-line continuous turbidity monitoring of filtrate from each membrane train or cassette must be provided for operational control and indirect membrane integrity monitoring. An alarm must be provided to notify the wastewater treatment facility operator of turbidity greater than or equal to 1.0 nephelometric turbidity units (NTU).

(B) An owner must follow the manufacturer's recommended frequency for MBR component inspection, testing, and maintenance. The manufacturer's recommended inspection, testing, and maintenance procedures and frequencies must be included in the wastewater treatment facility's operation and maintenance manual.

(C) An owner must provide a wastewater treatment facility operator access to any specialized tool necessary for the operation or maintenance of an MBR system. A description of all specialized tools and instructions for their use must be included in the operation and maintenance manual for the wastewater treatment facility.

(3) Control instrumentation. A wastewater treatment facility must have the ability to operate in full manual mode.

(f) Chemical Use and Disposal.

(1) The chemicals used in treatment and maintenance must not harm the MBR system or interfere with treatment.

(2) The chemicals used in treatment and maintenance, including concentrations of the chemicals and chemical disposal methods, must be identified in the engineering report.

(g) Training.

(1) The individuals trained to operate an MBR system must be familiar with the sequencing and set points of all operations and actions typically controlled by automated systems and be able to identify and respond to irregularities.

(2) The operation and maintenance manual must include instructions on how to operate the MBR in manual mode.

(h) Warranty and Bonds.

(1) All membranes must have a warranty of at least five years.

(2) The executive director may require a performance bond that meets the requirements of §217.7(b)(2)(E) of this title.

§217.158. Solids Management.

(a) Solids Recycling and Monitoring.

(1) A return sludge system must operate as designed in all flow conditions.

(2) A monitoring and control system must provide a means to control return and waste sludge flows from each clarifier, to control return sludge flows into each aeration basin, to meter return sludge flows, and to measure waste sludge flows.

(b) Solids Wasting. The solids management system must be able to store and process the waste activated sludge under all flow conditions.

(c) Return Activated Sludge (RAS) Pump Design.

(1) A centrifugal sludge pump must have a positive suction head, unless the pump is self-priming.

(2) An airlift pump must comply with requirements of §217.162 of this title (relating to Air Lift Pump Design).

(3) An RAS system must have sufficient pumping units to maintain the maximum design return pumping rate with the largest single pumping unit out of service.

(d) Waste Activated Sludge Pump Design. A waste activated sludge pumping system requires at least two pumping units and must be sized to prevent excessive solids accumulation in the clarifiers.

(e) Sludge Piping System.

(1) The design of a sludge piping system must accommodate cleaning and flushing.

(2) The design of a sludge piping system must accommodate a minimum velocity of 2.0 feet per second at the maximum wasting rate to prevent solids from settling. The velocity in a sludge piping system must prevent scouring.

(3) A sludge pipe must have a minimum diameter of 4.0 inches.

§217.159. Process Control.

(a) Solids Retention Time Control.

(1) A wastewater treatment facility design must include equipment for a wastewater treatment facility operator to control the solids retention time (SRT) in the aeration tanks by wasting a measured volume of surplus activated sludge.

(2) The engineering report and the operation and maintenance manual must provide the formulas used for determining the SRT.

(3) The SRT required for nitrification applies to the aerobic portion of the wastewater treatment facility.

(b) Aeration System Control. Aeration system control must regulate the total amount of air supplied and how air is distributed to the aeration tanks.

(1) In order to conserve energy, a wastewater treatment facility design may provide the operator with the ability to adjust the airflow in proportion to the oxygen demand of the wastewater.

(2) If an adjustable type of airflow control is installed, the aeration equipment must be manually adjustable over the entire range of oxygen demands and must maintain solids in suspension.

(c) A wastewater treatment facility with a design flow greater than 0.4 million gallons per day must provide for totalized flow measurement of the return sludge and waste sludge discharges for process control.

§217.160. Operability and Maintenance Requirements.

(a) All equipment must either be able to operate at the temperature extremes of the wastewater treatment facility location, or must be located in a temperature controlled enclosure.

(b) All equipment must be accessible for inspection, maintenance, and operation.

(c) An enclosure that houses equipment must have sufficient clearance and working room to safely remove and reinstall equipment. The enclosure must be accessible to portable lifting devices or must be equipped with overhead lifting eyes, hoists, trolleys, or cranes to facilitate the safe removal of equipment.

§217.161. Electrical and Instrumentation Systems.

(a) All three-phase motors must have phase failure protection.

(b) Instrumentation and monitoring equipment must have power surge protection.

(c) A wastewater treatment facility must conduct fault monitoring to notify the operator of high wet well level, power interruption, disinfection failure, blower failure, clarifier failure, return sludge pumping failure, and any other conditions that the executive director may require an owner to monitor as a condition for project approval.

(d) For a wastewater treatment facility not staffed 24 hours per day, a telemetry with battery back-up or supervisory control and data acquisition system with battery backup must be able to notify an operator of a malfunction identified in subsection (c) of this section within one minute of the malfunction.

§217.162. Air Lift Pump Design.

(a) Required Calculations.

(1) The engineering report must include calculations to determine static and dynamic pressure head necessary for operation of each air lift pump. Static and dynamic pressure head requirements for the suction line must be considered in the calculations.

(2) Air lift pump systems must be designed to meet the range of required flow rates.

(3) The engineering report must include the number of air lift pumps required and the minimum and maximum:

(A) design airflow rate for each air lift pump;

(B) design flow rate for each air lift pump; and

(C) design flow rate for the system.

(b) Design.

(1) Air lift pumps may not be used to pump sludge from a primary clarifier.

(2) Air lift pumps must be at least three inches in diameter.

(3) The design air flow rates must be based on the actual submergence of an air lift pump.

(4) Blowers for an air lift pump system must be sized to provide the maximum design airflow rate required by the system.

(5) The discharge end of an air lift pump must prevent splashing.

(6) Air lift pumps located inside of a basin must allow for cleaning without removal from the basin.

(7) An air lift pump must provide a way to release trapped air.

(8) Air lift pump systems for return activated sludge, waste activated sludge, internal recycle, and other systems that affect overall process performance must have a minimum dynamic submergence of 60%.

(9) Air lift pump systems for scum removal, grit removal, or for transfers that do not affect overall process performance must have a minimum dynamic submergence of 50%.

(10) Air lift pump systems for return activated sludge, waste activated sludge, internal recycle, and other systems that affect overall process performance must provide a method for flow measurement using:

(A) an external box with a weir; or

(B) a rotameter, or other flow measurement device approved in writing by the executive director on the air line to each pump. When flow is measured on the air line, the engineering report must include a graph correlating the air flow rate in the air line to the liquid flow rate of the air lift pump.

(c) Redundancy.

(1) A backup pump or other means of transfer must be provided for each air lift pump system.

(2) Air lift systems for return activated sludge, waste activated sludge, and internal recycle must be able to operate at full capacity with the largest pumping unit out of service.

§217.163. Advanced Nutrient Removal Systems.

[(a) For a wastewater treatment facility designed to provide advanced nutrient removal, the engineering report must specify the process units needed to achieve the effluent limits established in the wastewater treatment facility's wastewater permit.

(b) Biological nutrient removal using an activated sludge process, membrane filtration, sand filtration, or a combination of these processes may be used for advanced nutrient removal without applying for the executive director's approval under the innovative or non-conforming technology criteria in §217.7(b)(2) of this title (relating to Types of Plans and Specifications Approvals). A biological nutrient removal process that involves fixed-film treatment is subject to the executive director's determination under §217.7(b)(2) of this title.

(c) If a biological nutrient removal unit is proposed, the engineering report must include the:

(1) anticipated food to microorganism ratio in both the anoxic and anaerobic zones;

(2) volatile fatty acid recycle ratio; and

(3) design of a foaming control system.

(d) If a chemical addition unit is proposed, it must comply with the chemical containment requirements in Subchapter K of this chapter (relating to Chemical

Disinfection). Chemical addition may only be used if approved in writing by the executive director.]

(a) This section sets the minimum design requirements for biological and chemical nutrient removal systems. This section deals with nitrogen and phosphorus removal.

(b) Definitions

(1) Anaerobic zone -- The biological nutrient removal zone in which free, dissolved, or combined oxygen are not available.

(2) Anoxic zone -- The biological nutrient removal zone in which oxygen is only available in a combined form, such as nitrate (NO_3) or nitrite (NO_2).

(3) Aerobic zone -- The biological nutrient removal zone in which free and dissolved oxygen is available.

(4) Biological nitrogen removal -- The two-step biological process for the removal of nitrogen that is accomplished by the oxidation of ammonia to nitrite/nitrate (nitrification) in an aerobic environment followed by the reduction of nitrite/nitrate to nitrogen gas (denitrification) in an anoxic environment.

(5) Denitrification -- The biological process for the reduction of nitrate or nitrite to nitrogen gas or other products by denitrifying bacteria. Denitrification requires the presence of both a degradable carbon source and nitrate. Denitrification generates alkalinity and reduces oxygen consumption.

(6) Enhanced biological phosphorus removal (EBPR) -- The biological process for the removal of phosphorus by storage in heterotrophic phosphorus accumulating organisms selected in anaerobic/aerobic zones.

(7) Nitrification --The biological process for the oxidation of ammonia to nitrite and nitrate by nitrifying bacteria. Nitrification consumes alkalinity.

(8) Phosphorus accumulating organisms (PAOs) -- Heterotrophic bacteria in EBPR that have the ability for phosphorus storage.

(9) Postanoxic denitrification process -- A biological nitrogen removal process where the anoxic zone is located downstream of an aerobic nitrification zone

(10) Preanoxic denitrification process -- A biological nitrogen removal process where the anoxic zone is located upstream of an aerobic nitrification zone

(11) Readily biodegradable chemical oxygen demand (rbCOD) -- Dissolved biodegradable COD that is removed by bacteria much faster than colloidal or particulate degradable COD. It affects soluble phosphorus removal rates.

(12) $_{S}$BOD – Soluble 5-d biochemical oxygen demand

(13) SNdN -- Simultaneous nitrification-denitrification that is achieved in a single-reactor activated sludge system at a low DO concentration

(14) Cyclic NdN -- Cyclic nitrification-denitrification that is achieved in a single-reactor activated sludge system

(15) Supplemental Carbon -- Internal or external carbon source to support nitrogen and/or phosphorus biological removal processes

(16) Volatile Fatty Acids (VFAs) -- Specific organic carbon taken up by PAOs.

(c) Nutrient Removal Systems General Requirements

(1) For a wastewater treatment facility designed to provide biological nutrient removal (BNR), the engineering report must specify the process units needed to achieve the effluent limits established in the wastewater treatment facility's wastewater permit.

(2) A biological nutrient removal system that includes an activated sludge process with anoxic and aerobic zones, and anaerobic zone (if necessary), followed by secondary clarification and tertiary filtration may be used for biological nutrient removal without applying for the executive director's approval under the innovative or non-conforming technology criteria in §217.7(b)(2) of this title (relating to Types of Plans and Specifications Approvals).

(3) A secondary clarifier associated with an activated sludge process for biological nutrient removal must be designed according to §217.152, §217.153,

§217.154, and §217.170 of this Subchapter.

(4) A tertiary filter that follows an activated sludge system for biological nutrient removal must be designed according to §217.190, §217.191, §217.192, and §217.193 of Subchapter G of this title.

(5) The aeration and mixing equipment associated with an activated sludge process for biological nutrient removal must be designed according to §217.155 of this Subchapter.

(6) A biological nutrient removal process that involves fixed-film treatment or a combination of fixed-film and suspended growth treatment is subject to the executive director's determination under §217.7(b)(2) of this title.

(7) An existing wastewater treatment facility using a secondary activated sludge/final clarifier process that is upgraded for biological nutrient removal must be designed according to the requirements of this section.

(8) A biological nutrient removal system that involves an activated sludge membrane bioreactor (MBR) process must be designed according to §217.157 and the requirements of this section.

(9) A biological phosphorus removal system must be designed with a backup chemical precipitation system. The design must include provisions to manage the increase in sludge production due to the addition of chemicals (precipitants).

(10) The biological nutrient removal system design must provide redundancy for pumping systems for internal mixed liquor recycle (nitrate recycle) and RAS systems (n+1).

(11) A design of a wastewater treatment facility with a nutrient removal must include a foaming control system.

(12) The design of a biological nutrient system may include baffles to separate unaerated and aerated zones within a reactor, to trap foam, and to reduce the amount of oxygen recycled from the aerobic to the anoxic zone.

(13) Mixing. Unaerated (preanoxic, postanoxic, and anaerobic) zones must have a mechanical submersible mixing system or an alternative mixing system that has been approved in writing by the executive director. The use of coarse bubble air diffusers for mixing in a preanoxic tank will be reviewed on a

case-by-case basis.

(14) If a chemical addition unit is proposed, it must comply with the requirements in this section and the safety and chemical handling requirements in Subchapters K and M of this title. Chemical addition may only be used if approved in writing by the executive director.

(15) Supplemental carbon facilities. The need of a supplemental carbon facility must be evaluated based on the availability of readily biodegradable carbon concentration in the influent wastewater.

(d) Enhanced Biological Phosphorus Removal

(1) This subsection outlines the design requirements for enhanced biological phosphorus removal (EBPR) and for combined biological nitrogen and phosphorus removal systems. The design requirements for phosphorus removal by chemical precipitation are outlined in a separate subsection below.

(2) An EBPR system design must include anaerobic and aerobic zones, and anoxic zones (if combined nitrogen and phosphorus removal is required).

(3) The engineer shall submit calculations to support the sizing of the anaerobic and aerobic zones, and the anoxic zones (if combined nitrogen and phosphorus removal is required).

(4) The design of an EBPR system must ensure that readily biodegradable organic matter is available in the anaerobic zone to promote the growth of the phosphorus accumulating organisms (PAOs). The amount of readily biodegradable organic matter can be increased by fermentation of influent wastewater (primary sludge), fermentation of RAS or the addition of a supplemental external carbon source such as acetic acid.

(5) The anaerobic zone must be sized to accommodate PAO selection, VFA production, and RAS denitrification (in nitrifying EBPR systems).

(6) The design of the EBPR anaerobic zone must ensure that oxygen and nitrate are excluded or minimized to truly create anaerobic conditions. EBPR configurations in combined nitrogen and phosphorus removal systems include denitrification processes that limit the nitrate that is recycled to anaerobic zones.

(7) Sludge containing the phosphorus from the PAOs organisms can be wasted, or the phosphorus can be removed and/or recovered by a sidestream treatment process.

(8) The use of tertiary filtration must be evaluated to further remove effluent solids and to achieve low effluent total phosphorus.

(9) The EBPR design must include chemical addition provisions to meet stringent effluent requirements or to provide process reliability.

(10) The EBPR must be designed with operational flexibility to allow facility operators to respond to changing operating conditions.

(11) Some of the known processes that can achieve EBPR are shown in the following Figures and described briefly below:

(A) Anaerobic/Oxic (A/O) Process. A process configuration consisting of an anaerobic zone followed by an aerobic zone. RAS is recycled to the anaerobic zone.

(B) Anaerobic/Anoxic/Oxic (A^2O) Process. A process flow configuration consisting of a three-stage anaerobic/anoxic/aerobic process. Mixed liquor is recycled from the aerobic zone to the anoxic zone for denitrification. RAS is recycled to the anaerobic zone.

(C) Five-Stage Modified Bardenpho Process. A process configuration consisting of a five-stage anaerobic/anoxic/aerobic/anoxic/aerobic process for combined phosphorus and nitrogen removal. Mixed liquor (nitrate) is recycled from the first aerobic zone to the preanoxic zone for denitrification. RAS is recycled to the anaerobic zone. Carbon addition to the second anoxic zone is optional

Figure 30 TAC 217.163(11)(A)

Anaerobic/Oxic (A/O) Process

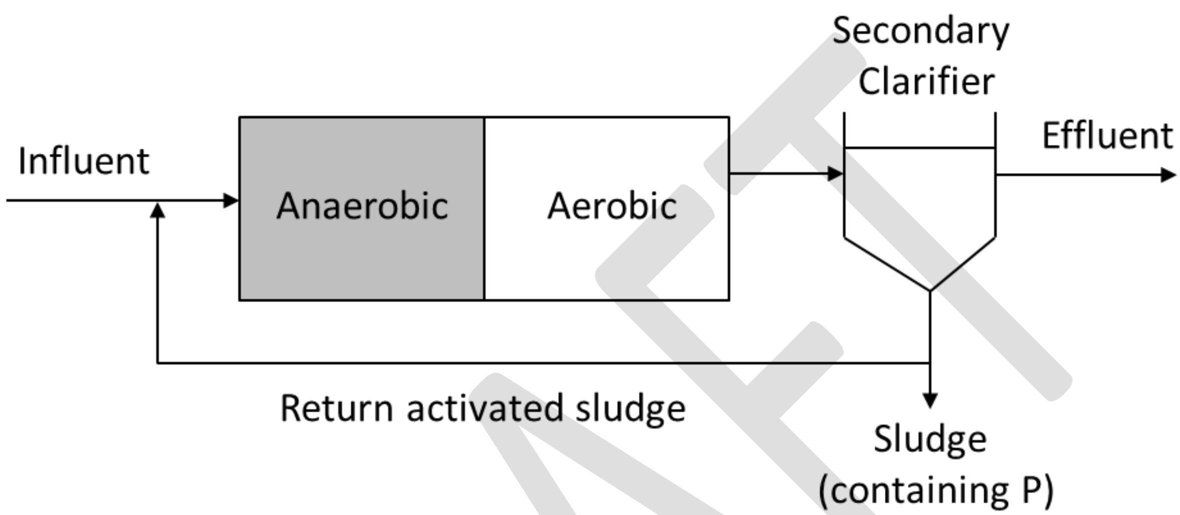


Figure 30 TAC 217.163(11)(B)

Anaerobic/Anoxic/Oxic (A²/O) Process

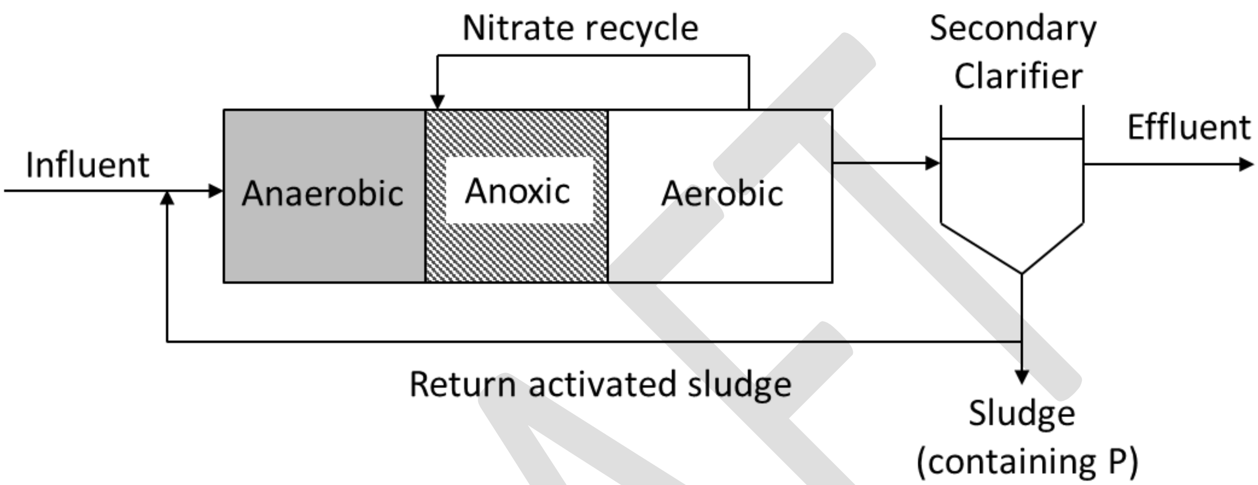
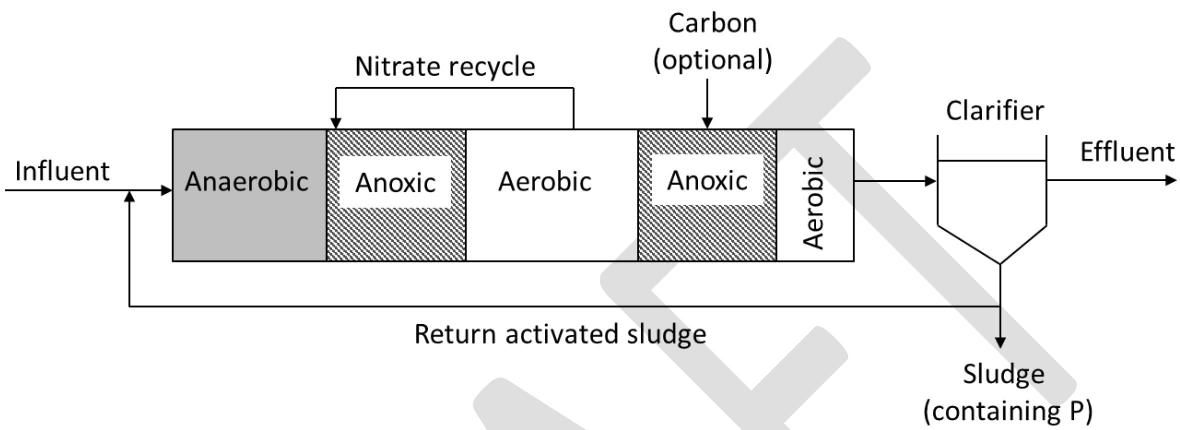


Figure 30 TAC 217.163(11)(C)

Five-Stage Modified Bardenpho Process



(D) University of Cape Town (UCT) Process. A process configuration consisting of a three-stage anaerobic/anoxic/aerobic process where both the RAS and aerobic tank contents are recycled to the anoxic zone, and biomass (mixed liquor) of the anoxic zone is recycled to the anaerobic zone. The internal recycle from the aerobic zone to the anoxic zone can be controlled, which allows that little nitrate be returned to the anaerobic tank. In the Modified University of Cape (MUCT) process, the anoxic zone is divided into two reactors. RAS enters the first anoxic reactor and internal recycle from the aerobic tank enters the second anoxic reactor. The MUCT process intends to eliminate nitrate recycle to the anaerobic tank while limiting the actual HRT in the anoxic zone to one hour.

(E) Johannesburg Process. The Johannesburg (JHB) process is a four-stage anoxic/anaerobic/anoxic/aerobic process characterized by the RAS denitrification (anoxic) zone ahead of three-stage anaerobic/anoxic/aerobic process. Internal recycle from the aerobic zone enters the second anoxic zone.

(F) Westbank Process. A process similar to the JHB process, which aims to improve denitrification rate by adding influent wastewater and RAS to the (preanoxic) denitrification zone. Some fractions of the influent wastewater may also be added to the downstream anaerobic and anoxic zones. The anaerobic zone is fed with supplemental carbon (VAFs) for PAO growth and efficient EPBR.

Figure 30 TAC 217.163(11)(D)

University of Cape Town (UCT) Process

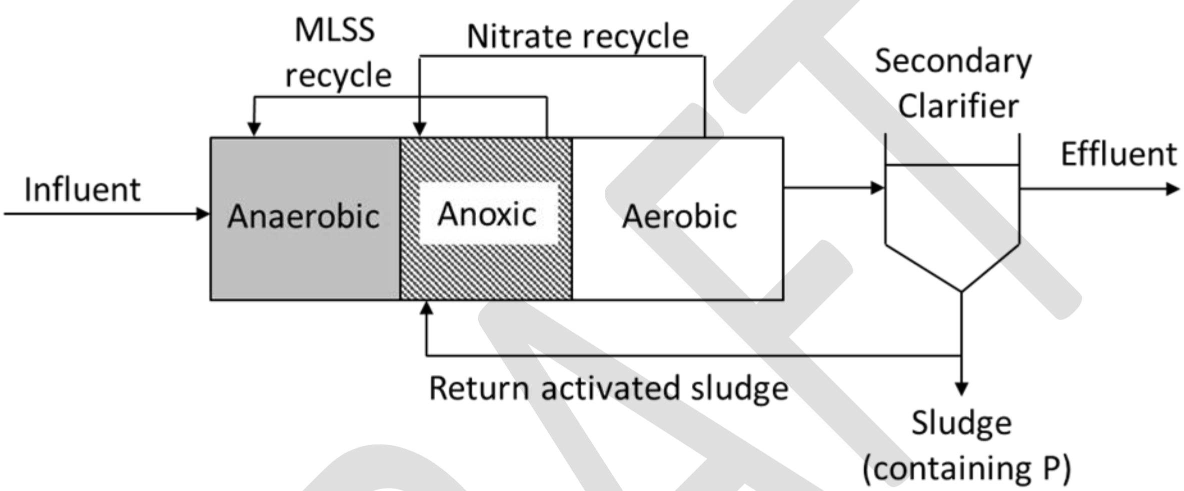


Figure 30 TAC 217.163(11)(E)

Johannesburg (JHB) Process

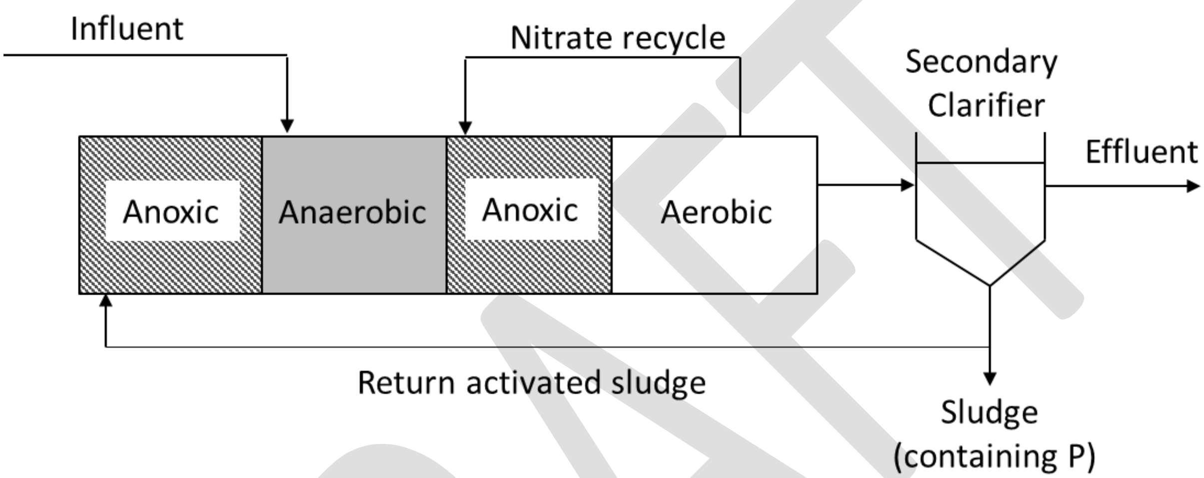
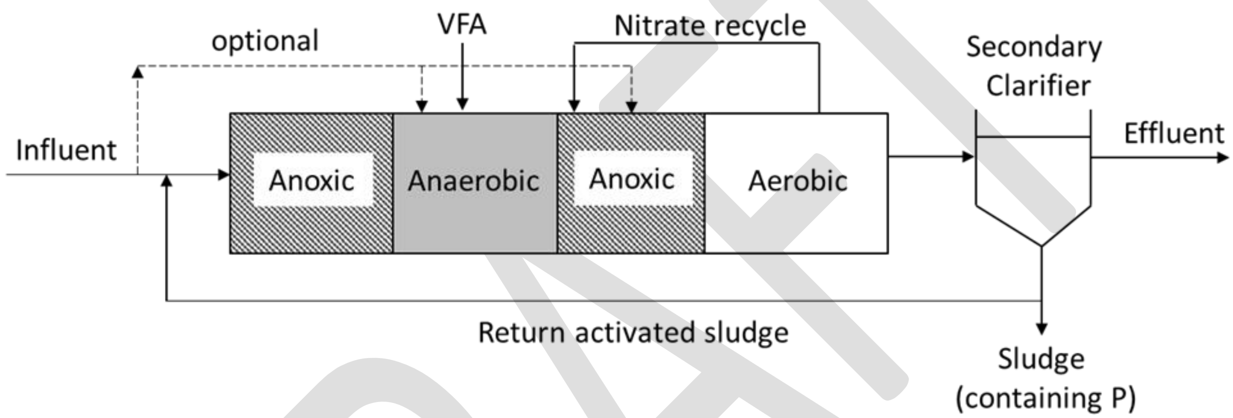


Figure 30 TAC 217.163(11)(F)

Westbank EBPR Process



(e) Enhanced Biological Phosphorus Removal Design Requirements

(1) The reactors volumes (zones sizing) must be determined using rate equations for substrate utilization and biomass growth (stoichiometric and kinetics parameters). For wastewater treatment plants that are designed with aerobic nitrification, the aerobic reactor must be designed according to Section 217.170 of this title, or another method approved by the executive director in writing.

(2) Modeling simulation packages can be used for the design of EBPR and combined nitrogen and phosphorus removal systems. However, it is recommended that preliminary sizing be accomplished using empirically derived design values (standard design parameters) followed by the use of modeling simulation packages for final process design and process design optimization.

(3) The EBPR system must be designed according to the following requirements:

(A) Influent Wastewater Characteristics. The engineering report must include the influent wastewater characteristics including the portion of the influent organic matter that is readily degradable, typically measured as rbCOD or VFAs and included in the engineering report. Table F.8 shows the minimum substrate to phosphorus ratios for design for EBPR (WEF 2018) .

Figure 30 TAC 217.163(e)(3)(A)

Table F.8 Minimum Substrate to Phosphorus Ratios for EBPR

Influent Substrate Parameter	Substrate to P ratio
$CBOD_5$	20:1
$sBOD_5$	15:1
COD	45:1
VFA	7:1 to 10:1
rbCOD	15:1

(B) SRT. For EBPR systems that are designed with nitrification, the minimum SRT must be at least the design SRT required for nitrification. The design SRT for an EBPR system that is not designed for nitrification must be at least 3 days but not more than 9 days. For EBPR systems that are designed for nitrification, the SRT must be at least 9 days.

(C) MLSS. The design operational range of MLSS for an EBPR system must be from 3,000 mg/L to 4,000 mg/L.

(D) Hydraulic Retention Time. The design hydraulic retention time (HRT) for an EBPR system anaerobic, anoxic, and aerobic zones must be:

(i) Anaerobic zone in the A/O, A^2O, 5-stage Bardenpho, JHB, and Westbank processes. At least 0.5 hr. but not more than 1.5 hr.

(ii) Anaerobic zone in the UCT process. At least 1.0 hr. but not more than 2.0 hr.

(iii) Anoxic zone in the A^2O and preanoxic zone in the 5-stage Bardenpho process. At least 1.0 hr. but not more than 3.0 hr.

(iv). Postanoxic zone in the 5-stage Bardenpho process and anoxic zones in the UCT, JHB and Westbank processs. At least 2 hr. but not more than 4.0 hr.

(v) Aerobic zone in the A/O process. At least 1 hr. but not more than 3.0 hr.

(vi) Aerobic zone in the A^2/O process. At least 4 hr. but not more than 8.0 hr.

(vii) First aerobic zone in the 5-stage Bardenpho process, and aerobic zones in the UCT, JHB and Westbank processes. At least 4 hr. but not more than 12.0 hr.

(viii) Second aerobic zone in the 5-stage Bardenpho process. At least 0.5 hr. but not more than 1 hr.

(E) Aeration (DO Limits). The oxygen concentration range for sizing aeration systems for EBPR zones must be:

(i) Anaerobic Zone. Not more than 0.2 mg/L for anaerobic zones.

(ii) Anoxic Zones. Not more than 0.5 mg/L for anoxic zones

(iii) Aerobic zones. At least 1.5 mg/L but not more than 3.0 mg/L for aerobic zones.

(F) Internal Recycle Rates. An EBPR system must be designed with internal recycle rates sufficient to minimize the nitrate concentration and to sustain the designed mixed liquor concentrations in the aeration, anoxic, and anaerobic zones.

(i) Anoxic Recycle (from aerobic to anoxic zone). The aerobic to anoxic recycle must be at least 100% but not more than 400% of the influent flow.

(ii) Anaerobic Recycle (from anoxic to anaerobic zone). The anoxic to anaerobic recycle must be at least 50% but not more than 100% of the influent flow.

(f) Operational Factors that affect EBPR

(1) Nitrate and Oxygen Addition to Anaerobic Contact Zone. Nitrate and DO in the process influent and recycle flows must be avoided or minimized where possible.

(2) Waste sludge processing. The design must consider the waste sludge

processing methods and the potential to recycle excessive amounts of phosphorus back to the EBPR process.

(3) pH and Alkalinity. Adjustment of pH to at least 7.0 would be beneficial to maximize EBPR performance. The need for alkalinity addition must be evaluated by conducting an alkalinity balance that considers the influent alkalinity concentration, alkalinity consumed by nitrification, and the alkalinity produced by denitrification.

(g) Phosphorus Removal by Chemical Precipitation

(1) A wastewater treatment facility that is designed with a EBPR process must include provisions for phosphorus removal by chemical precipitation.

(2) The design of an EBPR system with chemical precipitation can include alum salts, iron salts (ferric chloride), or lime as precipitating chemicals. Polymers can also be used in conjunction with metal salts and lime as flocculant aids.

(3) The chemical precipitation system must be designed with operational

flexibility to allow for adjustments in chemical feed points, chemical flowrates, or for alternate compounds.

(4) Process Design Requirements

(A) Dosage. Chemical dosages must be determined using the stoichiometry of the reactions and laboratory jar tests.

(B) Application Points. Chemical addition can be accomplished by a single point or multiple points of application. Typical points of application include:

(i) Primary clarifier

(ii) Secondary bioreactor

(iii) Secondary clarifier

(iv) Tertiary filter

(C) Chemical Feed System. The engineering report must include design information on each of the following components:

(i) Feeder

(ii) Dosage control

(iii) Storage.

(iv) Chemical Feed Lines

(v) Rapid mixing system

(5) Safety and Hazardous Chemical Handling. A phosphorus removal chemical precipitation system must be designed according to the safety and hazardous operation and maintenance requirements as set forth in §217.247, §217.280, and Subchapter M of this title.

(m) Biological Nitrogen Removal

(1) Biological nitrogen removal can be accomplished by biological nitrification of ammonia to nitrite/nitrate followed by biological denitrification

of nitrite/nitrate to nitrogen gas.

(2) A system designed for biological nitrogen removal with preanoxic/postanoxic denitrification processes must include isolated tanks, compartments, or baffled zones to separate anoxic and aerobic treatment zones

(3) The engineer shall submit calculations to support the sizing of the anoxic and aerobic reactor volumes.

(4) Commonly used activated sludge process configurations for biological nitrogen removal such as the Modified Ludzack Ettinger (MLE), Four-Stage Bardenpho™, and Step Feed BNR processes, which are shown in the following Figures, are not considered innovative and non-conforming technologies as defined in §217.7(b)(2).

Figure 30 TAC 217.163(m)(4)

Modified Ludzack-Ettinger (MLE) Process

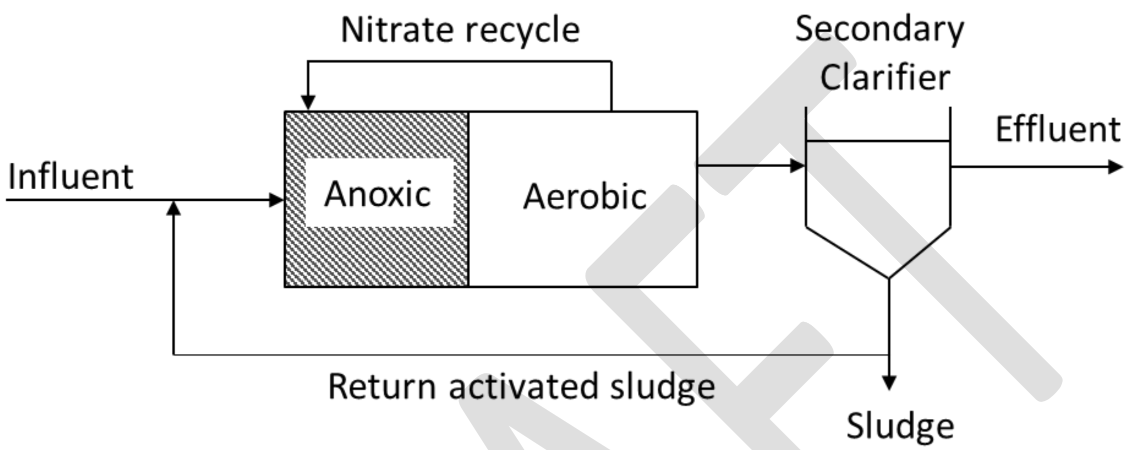


Figure 30 TAC 217.163(m)(4)

Four-Stage Bardenpho Process

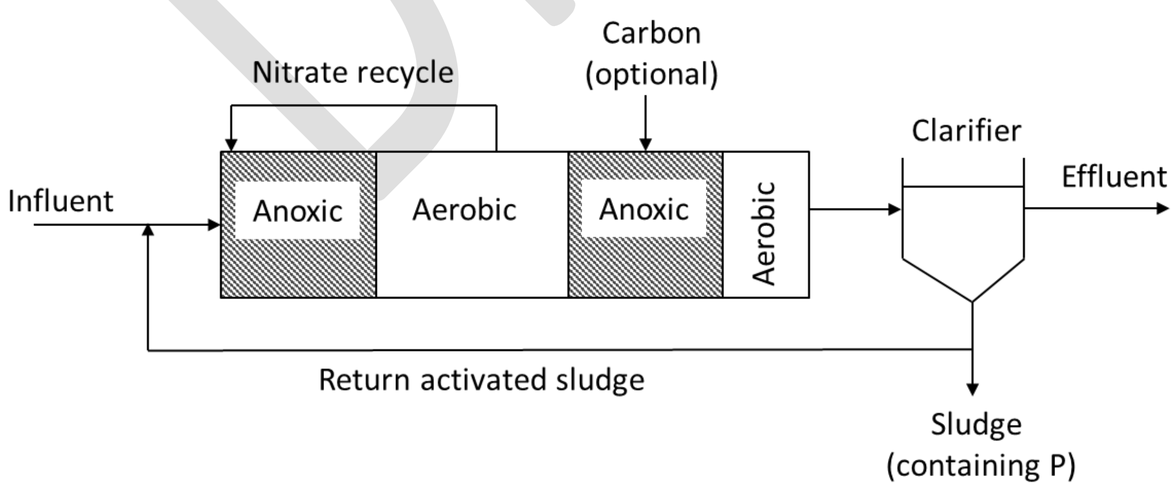
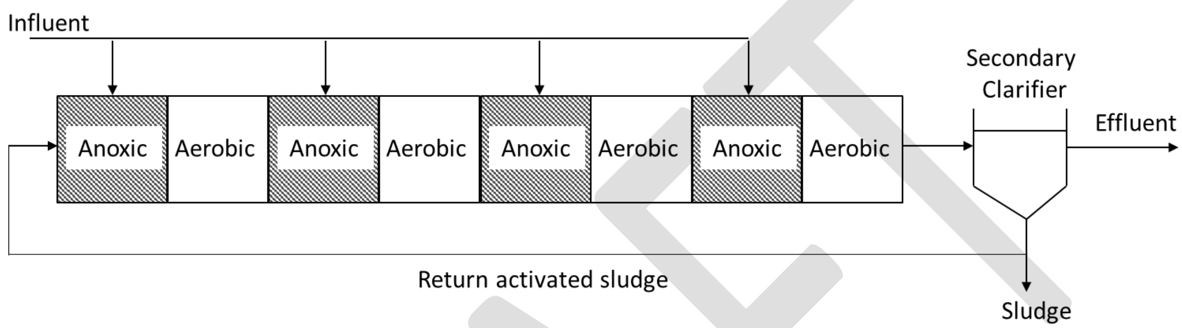


Figure 30 TAC 217.163(m)(4)

Step Feed BNR Process



(5) Activated sludge systems designed for biological nitrogen removal that include simultaneous nitrification-denitrification (SNdN) such as the oxidation ditch or Cyclic NdN processes such as SBRs must be approved on a case-by-case basis.

(n) Biological Nitrogen Removal System Design Requirements

(1) The reactors volumes (zones sizing) must be determined using rate equations for substrate utilization and biomass growth (stoichiometric and kinetics parameters). The aerobic reactor must be designed according to

Section 217.170 of this title, or another method approved by the executive director in writing.

(2) Modeling simulation packages can be used for the design of biological nitrogen removal systems. However, it is recommended that preliminary sizing be accomplished using empirically derived design values (standard design parameters) followed by the use of modeling simulation packages for final process design and process design optimization.

(3) Aerobic nitrification basin design. The aerobic reactor volume must be determined based on an adequate aerobic SRT for nitrification. The aerobic nitrification reactor must be designed according to the requirements in section 217.170 of this title.

(4) Anoxic denitrification basin design. The following factors must be considered in the design of the anoxic denitrification basin: SRT, influent wastewater characteristics, denitrification kinetics (specific denitrification rates), MLSS, mixed liquor internal recycle rates, RAS rates, HRT.

(5) Biological denitrification kinetics are affected by carbon substrate concentration, DO concentration, alkalinity and pH, and wastewater

temperature

(6) Because denitrification generates alkalinity and may reduce oxygen consumption; the system must be designed to benefit from this according to the following equivalences:

(A) Denitrification generates alkalinity. Alkalinity equivalency is 3.57 mg $CaCO_3$/ mg NO_3-N reduced to N_2

(B) Oxygen equivalency is 2.856 O_2/mg NO_3-N reduced to N_2

(7) The biological nitrogen removal system must be designed according to the following requirements:

(A) Influent Wastewater Characteristics. The engineering report must include the influent wastewater characteristics. Because nitrate reduction is dependent on having a sufficient degradable carbon source concentration relative to the amount of nitrogen to be removed, a minimum influent BOD to TKN of 4:1 is required to provide a sufficient amount of BOD.

(B) SRT. The minimum total SRT for a biological nitrogen removal system must be at least the design SRT required for nitrification. The design range SRT for the most common process configurations shall be:

(i) Preanoxic zone process (MLE process). At least 7 days but not more than 20 days.

(ii) Preanoxic and Postanoxic zone processes (4-stage Bardenpho) process. At least 5 days but not more than 20 days.

(iii) Oxidation ditch. At least 20 days but not more than 30 days

(iv) SBR. At least 10 days but not more than 30 days.

(C) MLSS. The design operational range of MLSS for a biological nitrogen removal system must be at least 2,000 mg/L but not more than 5,000 mg/L.

(D) Hydraulic Retention Time. The design hydraulic retention time (HRT) for a nitrogen removal system anoxic and aerobic zone must be:

(i) Anoxic zone in single stage preanoxic processes (MLE process). At least 1 hr. but not more than 3 hrs.

(ii) Anoxic zones in multi-stage preanoxic/postanoxic processes (4-stage Bardenpho process)

(I) Preanoxic zone. At least 1 hr. but not more than 3 hr.

(II) Postanoxic zone. At least 2 hr. but not more than 4 hr.

(iii) Aerobic zone in single stage processes (MLE Process). At least 4 hr. but not more than 12 hrs.

(iv) Aerobic zone in multi-stage processes (4-stage Bardenpho process).

(I) Aerobic zone downstream of preanoxic zone. At least 4 hr. but not more than 12 hr.

(II) Aerobic zone downstream of postanoxic zone. At least 0.5 hr but not more than 1 hr.

(v) Oxidation ditch. At least 18 hrs. but not more than 30 hrs.

(vi) SBR. At least 18 hrs. but not more than 30 hrs.

(E) Aeration (DO Limits). The design oxygen concentration range for sizing aeration or mixing systems for nitrogen removal systems must be:

(i) Anoxic Zones. Not more than 0.5 mg/L for anoxic zones

(ii) Aerobic zones. At least 1.5 mg/L but not more than 3.0 mg/L for aerobic zones.

(F) Internal Recycle Rates. A nitrogen removal system must be designed with internal recycle rates sufficient to minimize the nitrate concentration and to sustain the designed mixed liquor concentrations in the aeration and anoxic zones. The design range of aerobic to anoxic zone recycle must be at least 100% but not more than 400% of the influent flow.

(o) Operations and process control. A biological nitrogen removal facility must include equipment and instrumentation to monitor at least the following operating parameters: DO concentration, pH, alkalinity, temperature, nitrates, ammonia-nitrogen, and to control the SRT in the aeration tanks by wasting a measured volume of surplus activated sludge.

Removing §217.164 which will be updated and replaced by §217.170

[**§217.164. Aeration Basin and Clarifier Sizing--Volume-Flux Design Method.**

(a) A volume-flux design must size an aeration basin and clarifier on the relationship between the volume-flux of solids in the secondary clarifier, the sludge volume index (SVI), and the sludge blanket depth. The following design approach may be used as an alternative to the traditional design approach. If the volume-flux design approach is used, it must be used consistently throughout the design. No other design method may be used in combination with the volume-flux design method.

(1) A design may base the aeration tank volume and the clarifier volume on a mixed liquor suspended solids (MLSS) and floc volume (at SVI of 100) for the required minimum solids retention time.

(2) Larger values of MLSS require less aeration tank volume and greater clarifier volume.

(3) By examining a range of values of the MLSS and the floc volume, the most favorable arrangement for a wastewater treatment facility may be selected.

(4) When using the volume-flux design method, the size of an aeration basin and a clarifier must be in accordance with the requirements of this section.

(b) Design approach.

(1) Determine the solids retention time (SRT) needed to meet the permit requirement for five-day biochemical oxygen demand (BOD_5) and ammonia-nitrogen (NH3 -N) effluent limitations.

(2) Select a trial value mixed liquor floc volume (for example, MLSS at an SVI of 100).

(3) Using the design organic loading rate, the required SRT and yield, and the trial MLSS, determine the aeration tank volume.

(4) Using the trial value of mixed liquor flow volume, determine the clarifier area.

(5) For clarifiers overloaded in thickening at the peak flow, determine the final MLSS during storm flow and the resulting sludge blanket depth.

(6) Observing effluent limitations, determine the side water depth (SWD) and volume of the clarifier.

(7) Repeat the steps in paragraphs (2) - (6) of this subsection at different mixed liquor floc volumes and select the most favorable conditions for the wastewater treatment facility design.

(c) Aeration Basin Sizing.

(1) For a wastewater treatment facility that does not require nitrification, the minimum SRT is as follows:

(A) for a wastewater treatment facility with an effluent BOD_5 monthly average limitation of 20 milligrams per liter (mg/l), the minimum SRT is three days;

(B) for an extended aeration wastewater treatment facility with an effluent BOD_5 monthly average limitation of 20 mg/l, the minimum SRT is 22 days;

(C) for a wastewater treatment facility with an effluent BOD_5 monthly average limitation less than 20 mg/l, the minimum SRT is 4.5 days; and

(D) for an extended aeration wastewater treatment facility with an effluent BOD_5 monthly average limitation of less than 20 mg/l, the minimum SRT is 25 days.

(2) For a wastewater treatment facility that requires nitrification, the minimum SRT is based on the winter reactor temperature as set forth in §217.154(b) of this title (relating to Aeration Basin and Clarifier Sizing--Traditional Design) and the

values of SRT and net solids production (Y), as listed in Table F.8. in Figure: 30 TAC §217.164(c)(3). The maximum BOD_5 loading limitation for a single-step aeration process is 50 pounds (lb) BOD_5 per 1,000 cubic feet (cf) and for the first step of multi-step aeration process facilities is 100 lb BOD_5 /1,000 cf.

(3) An above-ground steel or fiberglass tank requires 2 degrees Celsius lower minimum operating temperature than a wastewater treatment facility utilizing a reinforced concrete tank. A wastewater treatment facility must be designed for an MLSS concentration of at least 2,000 mg/l but no more than 5,000 mg/l. The net solids production, (Y), in the following table includes both coefficients for yield and endogenous respiration:

Figure: 30 TAC §217.164(c)(3)

Table F.8. - Effect of Temperature on SRT, Net Solids Production, and Food to Mass Ratio			
Temperature, (degrees C)	**SRT, days**	**Net Solids Production, Y = .965-0.013(SRT)**	**Food/Mass Ratio, lbsBOD_5/lbs Suspend Solids/day = 1/(Y*SRT)**
18	4.76	0.90	0.233
17	5.25	0.90	0.212
16	5.79	0.89	0.194
15	6.38	0.88	0.178
14	7.04	0.87	0.163
13	7.77	0.86	0.150
12	8.56	0.85	0.137
11	9.45	0.84	0.126
10	10.42	0.83	0.116
This table uses the maximum growth rate of *Nitrosomonas* calculated using Equations 3-14 from EPA Manual, *Nitrogen Control*, EPA/625/R-93/010, 9/93, p. 90, shown in Figure: 30 TAC §217.164(c)(4), Equation F.5.			

(4) To calculate the SRT, divide the safety factor by the maximum growth rate as shown in the following equation. The safety factor includes the design factor for the ratio of average to maximum diurnal ammonia loading. A value of 3.0, as recommended in the United States Environmental Protection Agency manual Nitrogen Control, is used in calculating the values in Table F.8. in Figure: 30 TAC §217.164(c)(3).

Figure: 30 TAC §217.164(c)(4)

<u>Equation F.5.</u>

$$SRT = \frac{SF}{0.47\,e^{0.098(T-15)}}\text{, days}^{-1}$$

<u>Where:</u>

SF = safety factor

$0.47e^{0.098(T-15)}$ = maximum growth rate (days^{-1})

T = temperature T (°C)

(5) To determine the aeration basin volume, select a trial value of MLSS. The aeration basin volume is calculated as the maximum value from the following equations:

Figure: 30 TAC §217.164(c)(5)

Equation F.6.

$$V_a = \frac{1{,}000{,}000(BODL)(Y)(SRT)}{62.4MLSS}$$

Where:
V_a = Volume of aeration basin, cubic feet
BODL = Design biochemical oxygen demand (BOD) load per day, pounds
Y = yield of solids per unit BOD removed

SRT = required solids retention time, days
MLSS = mixed liquor suspended solids, milligrams per liter

Equation F.7.

$$\boldsymbol{V_a = \frac{1,000\ (BODL)}{max\ allowable\ lb\ BOD/kcf}}$$

Where:
V_a = Volume of aeration basin, cubic feet
BODL = Design BOD load per day, pounds
max allowable lb BOD/kcf = Maximum pounds BOD load/1,000 cubic feet

(d) Clarifier Sizing.

(1) A clarifier basin size is based on volume-flux from the floc volume of solids entering the clarifier.

(2) Biological solids may occupy different volumes for the same mass of solids as indicated by the SVI.

(3) For purposes of determining weir overflow rates for clarifier sizing, the design flow and the peak flow must include any return flows from units downstream of the clarifier, including flow from skimmers, thickeners, and filter backwash.

(4) A clarifier must be sized to prevent overloading under any design condition.

(5) The settling velocity of the mixed liquor solids must equal or exceed the two-hour peak weir overflow rate.

(6) A clarifier must be sized to prevent overloading in the thickening process at the design flow.

(7) The wastewater treatment facility's operation and maintenance manual must state the design maximum mixed liquor floc volume.

(e) Determine Weir Overflow Rate and Area. The values in Table F.9 in Figure 1: 30 TAC §217.164(e)(2)(I) determine the maximum surface loading rates. The MLSS concentration must include the same concentration used for sizing the aeration basin. The design must be based on the underflow rate. The design must include calculations for maximum weir overflow rate for the clarifier at the peak flow (Table F.9. in Figure 1: 30 TAC §217.164(e)(2)(I)), the aeration basin MLSS concentration, and a selected underflow rate. The area of the clarifier is determined by the following equation:

Figure: 30 TAC §217.164(e)

Equation F.8.

$$A_c = \frac{1,000,000\, Q_p}{OR_p}$$

Where:
A_c = area of the clarifier(s), square feet
Q_p = peak flow, million gallons per day
OR_p = weir overflow rate (gallons per day per square foot) from Table F.9. in Figure 1: 30 TAC §217.164(e)(2)(I)

(1) Determine Volume of a Clarifier. The volume of a clarifier must exceed the values determined from the minimum side wall depth (SWD) in Equation F.9. in Figure: 30 TAC §217.164(e)(1) or the minimum detention time in Equation F.10. in Figure: 30 TAC §217.164(e)(1):

Figure: 30 TAC §217.164(e)(1)

Equation F.9. Clarifier Volume Based on SWD

$$V_c = A_c\,(minSWD)$$

Where:
V_c = volume of the clarifier(s), cubic feet, based on minSWD
A_c = Area of the clarifier(s), square feet
minSWD = 10 feet, except as allowed in §217.152(g) of this title (relating to Requirements for Clarifiers)

Equation F.10. Clarifier Volume Based On Minimum Detention Time

$$V_c = \frac{(Q_p / 24)(minDT)}{(7.48)}$$

Where:
V_c = volume of the clarifier(s), cubic feet, based on minDT
Q_p = peak flow, gallons per day
minDT = minimum detention time (hours) from Table F.2. in Figure: 30 TAC §217.154(c)(1) of this title (relating to Aeration Basin and Clarifier Sizing--Traditional Design)

(2) Dimensions for Clarifiers Designed for Solids Storage Capabilities. The design of a clarifier that may be overloaded in thickening at the design flow must include the ability to store solids during peak flow events. The design must be based on the values in Table F.9. in Figure 1: 30 TAC §217.164(e)(2)(I), Table F.10. in Figure 2: 30 TAC §217.164(e)(2)(I), and Table F.11. in Figure 3: 30 TAC §217.164(e)(2)(I). The process for designing a clarifier based on this concept must be completed as follows:

(A) Determine the area of a clarifier. The area calculations must be based on the trial MLSS value selected for the sizing of the aeration basin in paragraph (1) of this subsection. The area of a clarifier must exceed the greater of the areas determined by Equation F.11. or Equation F.12. in Figure: 30 TAC §217.164(e)(2)(A):

Figure: 30 TAC §217.164(e)(2)(A)

Equation F.11. Clarifier Area Based on Design Flow

$$A_c = \frac{Q_d}{OR_{T9}}$$

Where:
A_c = clarifier area (square feet(sf)) based on max 30 day flow
Q_d = design flow (gallons per day)
OR_{T9} = weir overflow rate for selected underflow rate and mixed liquor suspended solids (MLSS) (gallons per day per square foot (gpd/sf)) from Table F.9. in Figure 1: 30 TAC §217.164(e)(2)(I)

Equation F.12. Clarifier Area Based on Peak Flow

$$A_c = \frac{Q_p}{OR_{T10}}$$

Where:
$A_{c,}$ = clarifier area (sf), based on peak flow
Q_p = peak flow, million gallons per day
OR_{T10} = weir overflow rate for selected MLSS (gpd/sf) from Table F.10. in Figure 2: 30 TAC §217.164(e)(2)(I)

(B) The final MLSS value must be the result of the transfer of solids from an aeration tank to a clarifier at the peak flow. A clarifier design must allow for rates of flow that will transfer solids from an aeration tank to a clarifier if the clarifier becomes overloaded in thickening until the mixed liquor solids are reduced to the concentration that no longer causes the overload.

(C) Using Table F.11. in Figure 3: 30 TAC §217.164(e)(2)(I) and the selected underflow rate, the MLSS concentration at peak flow is determined using the following equation:

Figure: 30 TAC §217.164(e)(2)(C)

Equation F.13.

$$MLSS_{pf} = \frac{UR_{T11} * RSSS_{T11}}{OR_{pf} + UR_{T11}}$$

Where:
UR_{T11} = Underflow rate (UR) (gallons per day per square foot (gpd/sf)) from Table F.11 in Figure 3: 30 TAC §217.164(e)(2)(I)
OR_{pf} = Weir overflow rate at peak flow (gpd/sf)
$MLSS_{pf}$ = Diluted mixed liquor suspended solids during peak flow (milligrams per liter (mg/l))
$RSSS_{T11}$ = Maximum return sludge concentration for the selected UR (mg/l) from Table F.11. in Figure 3: 30 TAC §217.164(e)(2)(I)

(D) Determine depth of sludge blanket at peak flow. The depth of a sludge blanket is determined by the aeration basin volume, the change in MLSS, the area of the clarifier, and the concentration of the blanket solids at the selected underflow rate as shown in the following equation:

Figure: 30 TAC §217.164(e)(2)(D)

Equation F.14.

$$SBD = \frac{V_a(MLSS_{av} - MLSS_{pf})}{(A_c BKSS)} + 1.0$$

Where:
SBD = Sludge Blanket Depth (feet)
V_a = Volume of aeration basins (cubic feet)
A_c = Area of clarifier (square feet)
$MLSS_{pf}$ = Diluted MLSS during peak flow (milligrams per liter (mg/l))
$MLSS_{av}$ = Diluted MLSS during average flow (mg/l)
BKSS = Blanket concentration at the selected underflow rate (mg/l) from Table F.11. in Figure 3: 30 TAC §217.164(e)(2)(I)
1.0 = Assumed sludge blanket depth during design flow conditions (feet)

(E) Determine the SWD. The SWD of a clarifier is the maximum value resulting from the following conditions:

(i) 10 ft, unless a lower depth is allowed by §217.152(g) of this title (relating to Requirements for Clarifiers);

(ii) 3.0 times the sludge blanket depth; and

(iii) minimum detention time per the following equation:

Figure: 30 TAC §217.164(e)(2)(E)(iii)

Equation F.15.

$$SWD_{DT} = \frac{OR_p(DT)}{180}$$

Where:
OR_p = Weir overflow rate at peak flow (gallons per day per square foot)
DT = Detention time, hours
SWD_{DT} = Side water depth based on detention time, feet

(F) Determine clarifier volume. The volume of a clarifier is the surface area multiplied by the SWD determined in subparagraph (E) of this paragraph.

Figure: 30 TAC §217.164(e)(2)(F)

Equation F.16.

$$V_c = A_c(SWD)$$

Where:
V_c = Volume of Clarifier, (cubic feet)
A_c = Area of the Clarifier, (square feet)
SWD = Side Water Depth determined in subparagraph (E) of this paragraph, (feet)

(G) The formulas for Equation F.17. in Figure: 30 TAC §217.164(e)(2)(G)(i); Equation F.18. in Figure: 30 TAC §217.164(e)(2)(G)(ii); and Table F.10. in Figure 2: 30 TAC §217.164(e)(2)(I); calculate the rates that are equal to the settling velocity of activated sludge at various floc volume concentrations. For values less than 30%, the floc volume is the 30-minute settled volume in an unstirred one-liter graduated cylinder. For values greater than 30%, the sample is diluted so that the settled volume is at least 15% but not more than 30%, and the result multiplied by the dilution factor.

(i) For floc volume less than 40%, use the following equation; or

Figure: 30 TAC §217.164(e)(2)(G)(i)

Equation F.17.

$$OR_{T10} = 5053.8(1 - fv/100)^{3.83}\ gpd/sf$$

Where:
OR_{T10} = Settling velocity (gallons per day per square foot) of Table F.10. in Figure 2: 30 TAC §217.164(e)(2)(I)
fv = Floc Volume (percent) = SVI(MLSS)/10,000 SVI (ml/g), MLSS (mg/l)

(ii) For floc volume greater than 40%, use the following equation:

Figure: 30 TAC §217.164(e)(2)(G)(ii)

Equation F.18.

$$OR_{T10} = 9003610(fv^{-2.56})gpd/sf$$

Where:
OR_{T10} = Settling velocity (gallons per day per square foot) of Table F.10. in Figure 2: 30 TAC §217.164(e)(2)(I)
fv = Floc Volume (percent) = SVI(MLSS)/10,000 SVI (ml/g), MLSS (mg/l)

(H) Table F.9. in Figure 1: 30 TAC §217.164(e)(2)(I) and Table F.11. in Figure 3: 30 TAC §217.164(e)(2)(I) are based on an analysis of the floc volume-flux, i.e. floc volume times settling velocity, calculated from Equation F.17. in Figure: 30 TAC §217.164(e)(2)(G)(i) and Equation F.18. in Figure: 30 TAC §217.164(e)(2)(G)(ii). Table F.11. in Figure 3: 30 TAC §217.164(e)(2)(I) is a tabulation of the maximum concentration of the underflow at different underflow rates. Equation F.19. is for Table F.11. in Figure 3: 30 TAC §217.164(e)(2)(I).

Figure: 30 TAC §217.164(e)(2)(H)

Equation F.19.

$$RSSS_m = \frac{10170000(UR^{-0.391})}{SVI}$$

Where:
$RSSS_m$ = Return Sludge Suspended Solids (milligrams per liter)
UR = Underflow Rate (gallons per day per square foot)
SVI = Sludge Volume Index (ml/g)

(I) Table F.9. in Figure 1: 30 TAC §217.164(e)(2)(I) determines the weir overflow rate that, along with the underflow rate and MLSS, determines the same floc volume-flux as shown in Table F.11. in Figure 3: 30 TAC §217.164(e)(2)(I).

Figure 1: 30 TAC §217.164(e)(2)(I)

Table F.9. - Clarifier Loading Rates

Allowable surface loading rates for given underflow rates with no provisions for sludge storage in the clarifier
OR
The maximum surface loading rate at the design flow for clarifiers designed to store solids during peak events

MLSS	**Underflow Rate (gpd/sf)**				
mg/l	***200***	***250***	***300***	***350***	***400***
2,000	1,081	1,218	1,340	1,452	1,554
2,100	1,020	1,148	1,262	1,366	1,461
2,200	965	1,084	1,191	1,288	1,377
2,300	914	1,026	1,126	1,217	1,299
2,400	868	973	1,067	1,151	1,229
2,500	825	924	1,012	1,091	1,163
2,600	786	879	962	1,036	1,103
2,700	749	837	915	985	1,048
2,800	715	798	872	937	996
2,900	684	762	831	893	948
3,000	654	729	793	851	903
3,100	627	697	758	812	861

3,200	601	667	725	776	821
3,300	577	640	694	742	784
3,400	554	613	665	710	750
3,500	532	589	637	680	717
3,750	483	533	575	611	642
4,000	441	484	520	551	577
4,250	403	441	472	498	520
4,500	369	402	429	451	469
4,750	340	368	391	409	423
5,000	313	337	356	371	382

Figure 2: 30 TAC §217.164(e)(2)(I)

Table F.10. - Settling Velocities

Maximum allowable clarifier weir overflow rate allowed for clarifiers that are designed to store solids SVI=100 *(minimum allowable SVI)*	
MLSS	**Surface Loading Rates**
(mg/l)	**(gpd/sf)**
2,000	2,000
2,150	2,000
2,200	1,952
2,300	1,858
2,400	1,767
2,500	1,680
2,600	1,596
2,700	1,514
2,800	1,437
2,900	1,362
3,000	1,290
3,100	1,220
3,200	1,154
3,300	1,090
3,400	1,029
3,500	971

3,750	836
4,000	715
4,250	611
4,500	528
4,750	459
5,000	403

Figure 3: 30 TAC §217.164(e)(2)(I)

Table F.11. - Values for Use in Determining Sludge Storage Requirements					
Underflow (gpd/sf)	200	250	300	350	400
RSSS maximum (mg/l)	12,813	11,743	10,935	10,295	9,771
Blanket concentration (mg/l)	7,816	7,163	6,670	6,280	5,961
Blanket (lb/cf)	0.488	0.447	0.416	0.392	0.372

]

§217.170. Aeration Basin and Clarifier Sizing-Kinetics-Based Approach.

(a) This section applies the activated sludge process kinetics approach to determine the aeration tank volume. The method is based on the calculation of the activated sludge solids retention time (SRT) as the main design parameter along with the use of kinetic relationships to determine the aeration tank volume as a function of SRT, Mixed Liquor Suspended Solids (MLSS) concentration and the observed yield solids production (Y). The clarifier area can be determined using the surface loading rates outlined in §217.154(c) of this title or using solids loading rates as determined by a solids flux analysis.

(b) Design approach.

(1) Determine the solids retention time (SRT) needed to meet the permit requirements for five-day biochemical oxygen demand (BOD_5), and ammonia-nitrogen (NH_3-N) effluent limitations.

(2) Select a trial value mixed liquor suspended solids (MLSS) concentration.

(3) Using the design organic loading rate, the required SRT, the net yield (Y), and the trial MLSS, determine the aeration tank volume.

(4) Determine the clarifier area using the Aeration Basin and Clarifier Sizing--Traditional Design method outlined in §217.154(c) of this subchapter or a solids flux analysis as described in Water Environment Federation (WEF) Manual of Practice (MOP) No.8, Sixth Edition, 2018 or Metcalf and Eddy’s Wastewater Engineering Treatment and Resource Recovery, Fifth Edition, 2014.

(5) Determine the side water depth (SWD) and volume of the clarifier.

(6) Using the Aeration Equipment Sizing method outlined in §217.155 of this subchapter, determine the oxygen requirements of wastewater and the airflow requirements for sizing the aeration system components.

(7) Repeat the steps in paragraphs (2) - (6) of this subsection at different mixed liquor floc concentrations and select the most favorable conditions for the wastewater treatment facility design.

(c) Aeration Basin Sizing.

(1) For a wastewater treatment facility that does not require nitrification the minimum SRT is as follows:

(A) for a wastewater treatment facility with an effluent BOD_5 monthly average limitation of 10 milligrams per liter (mg/L), the minimum SRT is three days.

(B) for an extended aeration wastewater treatment facility with an effluent BOD_5 monthly average limitation of 10 mg/L, the minimum SRT is 22 days;

(C) for a wastewater treatment facility with an effluent BOD_5 monthly average limitation less than 10 mg/l, the minimum SRT is 5-days; and

(D) for an extended aeration wastewater treatment facility with an effluent BOD5 monthly average limitation of less than 20 mg/l, the minimum SRT is 25days.

(2) For a wastewater treatment facility that requires nitrification, the SRT is based on the winter reactor temperature as set forth in §217.154(b) of this title (relating to Aeration Basin and Clarifier Sizing--Traditional Design) and the values of SRT and net solids production (Y), as determined below.

(3) A wastewater treatment facility must be designed for an MLSS concentration of at least 2,000 mg/l but no more than 5,000 mg/l.

(4) To calculate the SRT, determine the specific growth rate for the ammonia-oxidizing bacteria (μ_{AOB}) using Equation F.5. The specific growth rate for the ammonia-oxidizing bacteria (AOB) is a function of the maximum growth rate of ammonia-oxidizing bacteria ($\mu_{max\ AOB}$), ammonia nitrogen (NH_4-N) concentration, dissolved oxygen (DO) concentration and the endogenous decay rate of ammonia-oxidizing bacteria. The nitrification rate will control the design because the ammonia-oxidizing bacteria (AOB) grow more slowly than the heterotrophic bacteria that remove BOD_5.

Figure: 30 TAC §217.170(c)(4)

Equation F.5.

$$\mu AOB = \mu maxAOB\left(\frac{SNH}{SNH+KNH}\right)\left(\frac{So}{So+Ko\ AOB}\right) - bAOB$$

Where:

μ_{AOB} = specific growth rate of ammonia-oxidizing bacteria, g VSS/g VSS.d

$\mu_{max\ AOB}$ = maximum specific growth rate of ammonia-oxidizing bacteria, g VSS/g VSS.d

b_{AOB} = specific endogenous decay rate of ammonia-oxidizing bacteria, g VSS lost/g VSS.d

S_{NH} = Reactor NH_4-N concentration, mg/L

K_{NH} = half-velocity coefficient for NH_4-N, mg/L

S_O = Reactor DO concentration, mg/L, 2.0 mg/L for nitrification

K_{OAOB} = half-velocity coefficient for DO for AOB, mg/L

Select values for $\mu_{max\ AOB}$, b_{AOB}, K_{NH} and $K_{OAOB.}$ Typical kinetic coefficient values at 20°C are:

$\mu_{max\ AOB}$ = 0.90 g/g.d

b_{AOB} = 0.17 g/g.d

K_{NH} = 0.50 g/m3

K_{OAOB} = 0.50 g/m3

Using typical temperature correction θ values, calculate $\mu_{max\ AOB}$, and b_{AOB} at the design reactor temperature with Equations F.6. and F.7.

Equation F.6.

$\mu_{max\ AOB} = (\mu_{max\ AOB20})(\theta)$^T-20

Where:

$\mu_{max\ AOB}$ = maximum specific growth rate of ammonia-oxidizing bacteria at reactor temperature

$\mu_{max\ AOB20}$ = maximum specific growth rate of ammonia-oxidizing bacteria at 20°C, (typical value of 0.90 g/g.d)

θ = temperature correction value (typical value of 1.072)

T = reactor temperature

Equation F.7.

$b_{AOB} = (b_{AOB20})(\theta)^{T-20}$

Where:

b_{AOB} = specific endogenous decay rate of ammonia-oxidizing bacteria at reactor temperature, g VSS lost/g VSS.d

b_{AOB20} = specific endogenous decay rate of ammonia-oxidizing bacteria at 20°C, g VSS lost/g VSS.d. (typical value 0.17 g/g.d)

θ = temperature correction value (typical value of 1.029)

T = reactor temperature

Substitute the above values in Equation F.5. and solve for μ_{AOB}.

Determine the theoretical and design SRT with Equations F.8 and F.9., respectively

Equation F.8.

$$SRT = 1/(\mu AOB)$$

Where:

SRT = Solids retention time, d

μ_{AOB} = Specific growth rate of ammonia-oxidizing bacteria, g VSS/g VSS.d

Equation F.9.

$$SRTdesign = SRT\ (SF)$$

Where:

SF Safety Factor (can be based on expected peak/average TKN or ammonia loadings and may vary from 1.3 to 2.0).

(5) To determine the aeration basin volume, select a trial value of MLSS. The aeration basin volume is calculated as the maximum value from

Equations F.10 and F.11.

Equation F.10. (Aeration Basin Volume Based on SRT)

$$Va = 1{,}000{,}000\ (BODL)\ (Y_{obs})(SRT)\ /(62.4\ MLVSS)$$

Where:

Va = Volume of aeration basin, cubic feet

BODL = Design organic loading, pounds of BOD load per day, lb/d

SRT_{design} = Design solids retention time, days

MLVSS = Mixed liquor volatile suspended solids, milligrams per liter, mg/L (typically 80% of the trial value of MLSS)

Y_{obs} = Observed yield of solids per unit BOD removed. Observed solid yield (Y) values as a function of SRT and temperature are shown in Tables F.9 and F.10 for activated sludge systems with primary treatment and without primary treatment, respectively (adapted from WEF 2018, page 881).

Table F.9. Effect of SRT and Temperature on Net Solids Production (Y) with Primary Treatment

SRT (days)	Net Solids Production, Y (lb VSS/lb BOD.d)		
	10°C	20°C	30°C
3	0.77	0.67	0.56
4	0.74	0.64	0.54
5	0.71	0.61	0.52
6	0.68	0.59	0.50
7	0.67	0.58	0.48
8	0.65	0.56	0.46
9	0.63	0.55	0.45
10	0.61	0.53	0.44
15	0.55	0.47	0.41
20	0.49	0.43	0.38
25	0.47	0.41	0.36
30	0.43	0.38	--

Table F.10. Effect of SRT and Temperature on Net Solids Production (Y) without Primary Treatment

SRT (days)	Net Solids Production, Y (lb VSS/lb BOD d)		
	10°C	20°C	30°C
3	1.08	0.97	0.89
4	1.05	0.93	0.86
5	1.02	0.91	0.84
6	0.98	0.89	0.82
7	0.96	0.86	0.80
8	0.94	0.84	0.79
9	0.91	0.83	0.77
10	0.89	0.81	0.76
15	0.82	0.75	0.70
20	0.77	0.70	0.65
25	0.73	0.67	0.62

Equation F.11. (Aeration Volume Based on Maximum Organic Loading Rate)

$$Va = 1{,}000\ (BODL) / (\text{max allowable lb BOD/day/1000 ft}^3)$$

Where:

Va = Volume of aeration basin, cubic feet

BODL = Design organic loading, pounds of BOD per day, lb/d

Max allowable lb BOD/day/1000 ft^3 = Maximum allowable organic loading rate in pounds BOD/day/1,000 cubic feet, as requested in Table F.1. of this title

(d) Clarifier Sizing.

(1) For purposes of determining weir overflow surface loading rates for clarifier sizing, the design flow and the peak flow must include any return flows from units downstream of the clarifier, including flow from skimmers, thickeners, and filter backwash.

(2) A clarifier must be sized to prevent overloading under any design condition.

(3) The settling velocity of the mixed liquor solids must equal or exceed the two-hour peak surface loading rate.

(4) Determine the area of a clarifier. The area calculations must be based on the maximum surface loading rates at 2-hr peak flow, as outlined in Table F.2. or the solids loading rates (solids limiting rate) as determined by a solids flux analysis. The area of a clarifier must be based on the larger of the surface areas determined for surface loading rates and solids loading rates (solids flux analysis).

(5) Determine the side water depth (SWD). The SWD of a clarifier is the maximum value resulting from the following conditions:

(i) 10 ft, unless a lower depth is allowed by §217.152(g) of this title (relating to Requirements for Clarifiers)

(ii) 3.0 times the sludge blanket depth; and

(iii) Minimum detention time per ~~the following~~ equation F.12.

Equation F.12.

$$SWD_{DT} = SLRp\,(DT)/180$$

Where:

SLRp = Surface loading rate at peak flow (gallons per day per square foot)
DT = Detention time, hours
SWD_{DT} = Side water depth based on detention time, feet

(6) Determine the clarifier liquid volume per equation F.13. The volume of a clarifier is the surface area multiplied by the SWD determined in paragraph (5) above.

Equation F.13.

$$Vc = Ac\,(SWD)$$

Where:

Vc = Volume of Clarifier, (cubic feet)
Ac = Area of the Clarifier, the larger of the surface areas determined for solids loading rates in Table F.2 and solids flux analysis (square foot)
SWD = Side water Depth as determined in paragraph (5) above (feet)