Chapter seven

Design of wastewater treatment systems

Zeng Zhuo

Environmental Engineering

Southwest Jiaotong University

Outline

I. **Wastewater standards, design flows and loadings, primary treatment**
II. **Secondary wastewater treatment**
III. **Oxygen transfer and mixing**
IV. **Advanced biological treatment systems**
V. Attached-growth biological systems
VI. Secondary clarification
VII. Disinfection
VIII. Solids handling and treatment systems

Section 1

Wastewater standards,

design flows and loadings,

primary treatment

Wastewater standards

Wastewater treatment plants* = *WWTPs

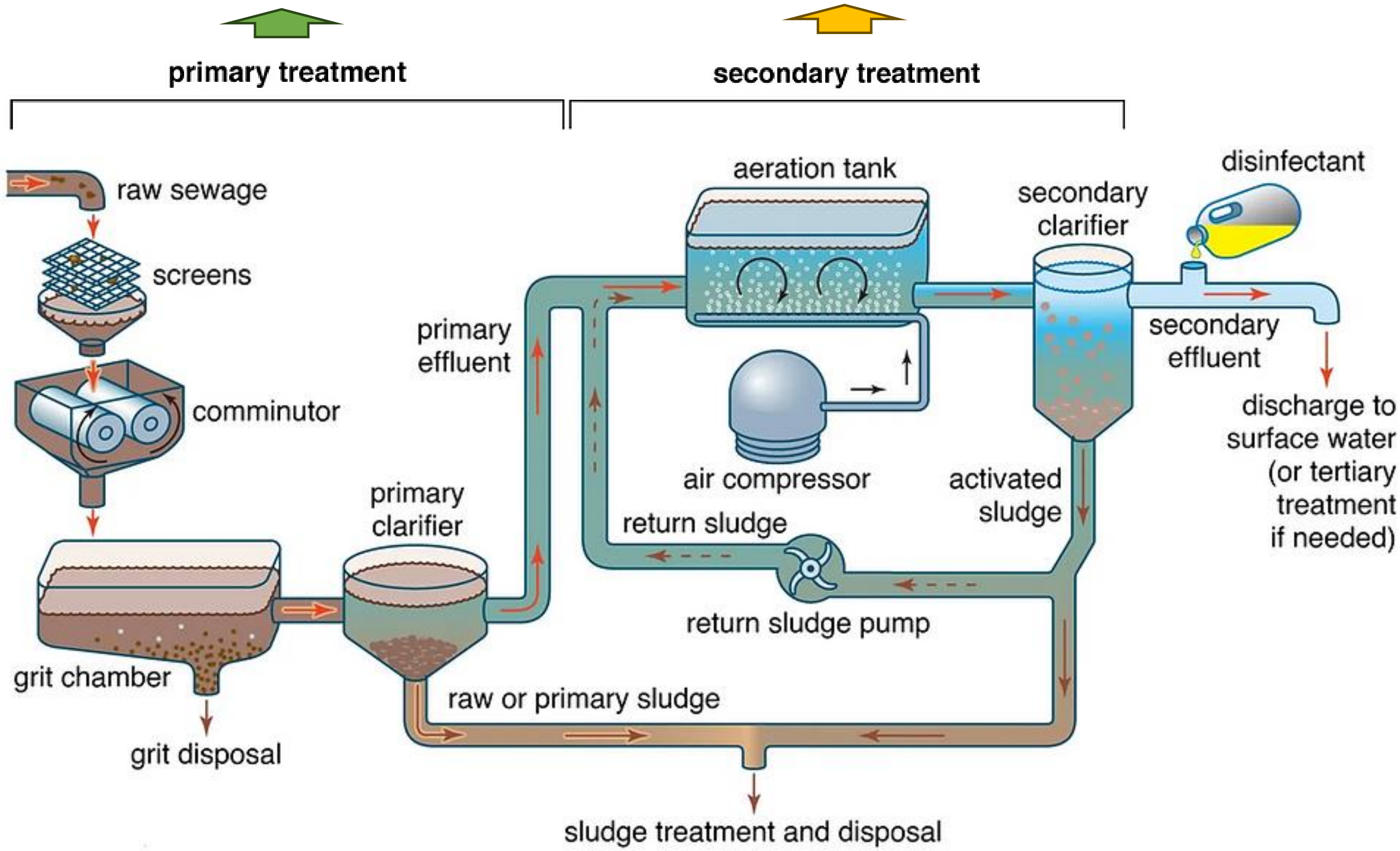
A unit operation is defined as a physical treatment such as screening, grit removal, sedimentation, and sludge dewatering.

A unit process is defined as a treatment that involves a biological or chemical reaction such as activated sludge, membrane bioreactors, and chlorination.

Wastewater standards

To remove particulate matter (total suspended solids and particulate organic matter)

To remove dissolved and colloidal organic matter (as measured by BOD_5) and total suspended solids (TSS), and kill pathogens.



Wastewater standards

Effluent standards (in U.S.)

The National Pollution Discharge Elimination System (NPDES)

Parameter	For secondary WWTPs			For advanced WWTPs (mg/L)
	30-day average (% removal)	30-day average (mg/L)	7-day average (mg/L)	
BOD_5	85	30	45	5
$CBOD_5$	85	25	40	-
TSS	85	30	45	5
pH	-	6.5-8.5	6.5-8.5	6.5-8.5
Total nitrogen (TN)	-	-	-	3
Total phosphorous (TP)	-	-	-	1

Wastewater standards

Effluent standards (in P.R.C)

Discharge standard of pollutants for municipal wastewater treatment plant (GB-18918-2002)

中 华 人 民 共 和 国 国 家 标 准

GB 18918—2002

城镇污水处理厂污染物排放标准

Discharge standard of pollutants for municipal wastewater treatment plant

2002—12—24 发布　　2003—07—01 实施

国 家 环 境 保 护 总 局
国家质量监督检验检疫总局 发布

表 1　基本控制项目最高允许排放浓度（日均值）　单位 mg/L

序号	基本控制项目		一级标准 A 标准	一级标准 B 标准	二级标准	三级标准
1	化学需氧量（COD）		50	60	100	120①
2	生化需氧量（BOD_5）		10	20	30	60①
3	悬浮物（SS）		10	20	30	50
4	动植物油		1	3	5	20
5	石油类		1	3	5	15
6	阴离子表面活性剂		0.5	1	2	5
7	总氮（以 N 计）		15	20	–	–
8	氨氮（以 N 计）②		5（8）	8（15）	25（30）	–
9	总磷（以 P 计）	2005 年 12 月 31 日前建设的	1	1.5	3	5
		2006 年 1 月 1 日起建设的	0.5	1	3	5
10	色度（稀释倍数）		30	30	40	50
11	pH		6-9			
12	粪大肠菌群数（个/L）		10^3	10^4	10^4	–

Design flows and loadings

Design flows

Annual average Daily Flow (ADF) = population × 0.46 m^3 per capita per day

Flow Ratios (Peaking Factors) for residential wastewater flows

Flow ratio	Range	Conditions
Annual average Daily Flow (ADF)	-	-
Maximum Month Flow(MMF): ADF	0.9:1 to 1.2:1	**The design plant capacity**
Minimum Month Flow: ADF	0.9:1 to 1.1:1	-
Peak Day Flow (PDF) : ADF	2:1	-
Peak Hour Flow (PHF) : ADF	3:1	**For pumps and pipes design**
Minimum Day Flow (MDF) : ADF	0.67:1	-
Minimum Hour Flow (MHF) : ADF	0.33:1	-

Design flows and loadings

Design loadings

Wastewater loadings ($\dot{m}$) refer to the mass concentration of a specific constituent multiplied by the volumetric flow rate.

$$\dot{\boldsymbol{m}}(\mathbf{kg/d}) = Q(\frac{\mathrm{m}^3}{\mathrm{d}}) \times C(\frac{\mathrm{mg}}{\mathrm{L}}) \times 1 \times 10^{-6}(\frac{\mathrm{kg}}{\mathrm{mg}}) \times 1 \times 10^{3}(\frac{\mathrm{L}}{\mathrm{m}^3})$$

Q=volumetric flow rate, m^3/d
C= concentration of constituent, mg/L
$\dot{m}$= mass loading rate of constituent, kg/d

Average daily loadings = Average Daily Flow × Average (typical) concentration

Peak daily loadings = Peaking factor × Average daily loadings

Typical concentration for dometic wastewater can see Table 7.4 in page 335.

Peaking factors for wastewater loading rates can see Table 7.5 in page 336.

Design flows and loadings

Example 7.1 Calculating design flows and mass loadings using literature values

A new residential development in Blacksburg, Virginia is expected to have a build-out population of 5000 people by the year 2020. Assume a domestic wastewater with characteristics from column 1 in Table 7.4 for the United States. Using the values in Table 7.2, estimate the following for the year 2020:

a) Average daily flow (m^3/d).
b) Peak hour flow (m^3/d).
c) Minimum hour flow (m^3/d).
d) Average daily BOD_5 mass loading (kg/d).
e) Average daily TSS mass loading (kg/d).
f) Average daily TN mass loading (kg/d).

a) Average daily flow = 5000×460 Lpcd = 2.3×10^6 L/d = $\mathbf{2.3 \times 10^3\ m^3/d}$
b) Peak hour flow= ratio $\times$ ADF = $3 \times 2.3 \times 10^3$ = $\mathbf{6.9 \times 10^3\ m^3/d}$
c) Minimum hour flow = ratio $\times$ ADD = $0.33 \times 2.3 \times 10^3$ = $\mathbf{7.59 \times 10^2\ m^3/d}$
d) Average daily BOD_5 mass loading = $2.3 \times 10^3 \times 200 \times 10^{-6} \times 10^3$= **460 kg$BOD_5$/d**
e) Average daily TSS mass loading =$2.3 \times 10^3 \times 240 \times 10^{-6} \times 10^3$= **552 kgTSS/d**
f) Average daily TN mass loading=$2.3 \times 10^3 \times 35 \times 10^{-6} \times 10^3$= **80.5 kgTN/d**

Primary treatment

Preliminary treatment of wastewater focuses primarily on physical treatment schemes, to remove large debris, paper, plastics, hair, grit, eggshells, coffee grinds, and sand.

- **Screening**
- **grit removal**
- flow measurement
- Pumping
- flow equalization
- pre-aeration

Primary treatment = Preliminary treatment + pre-sedimentation

Primary treatment

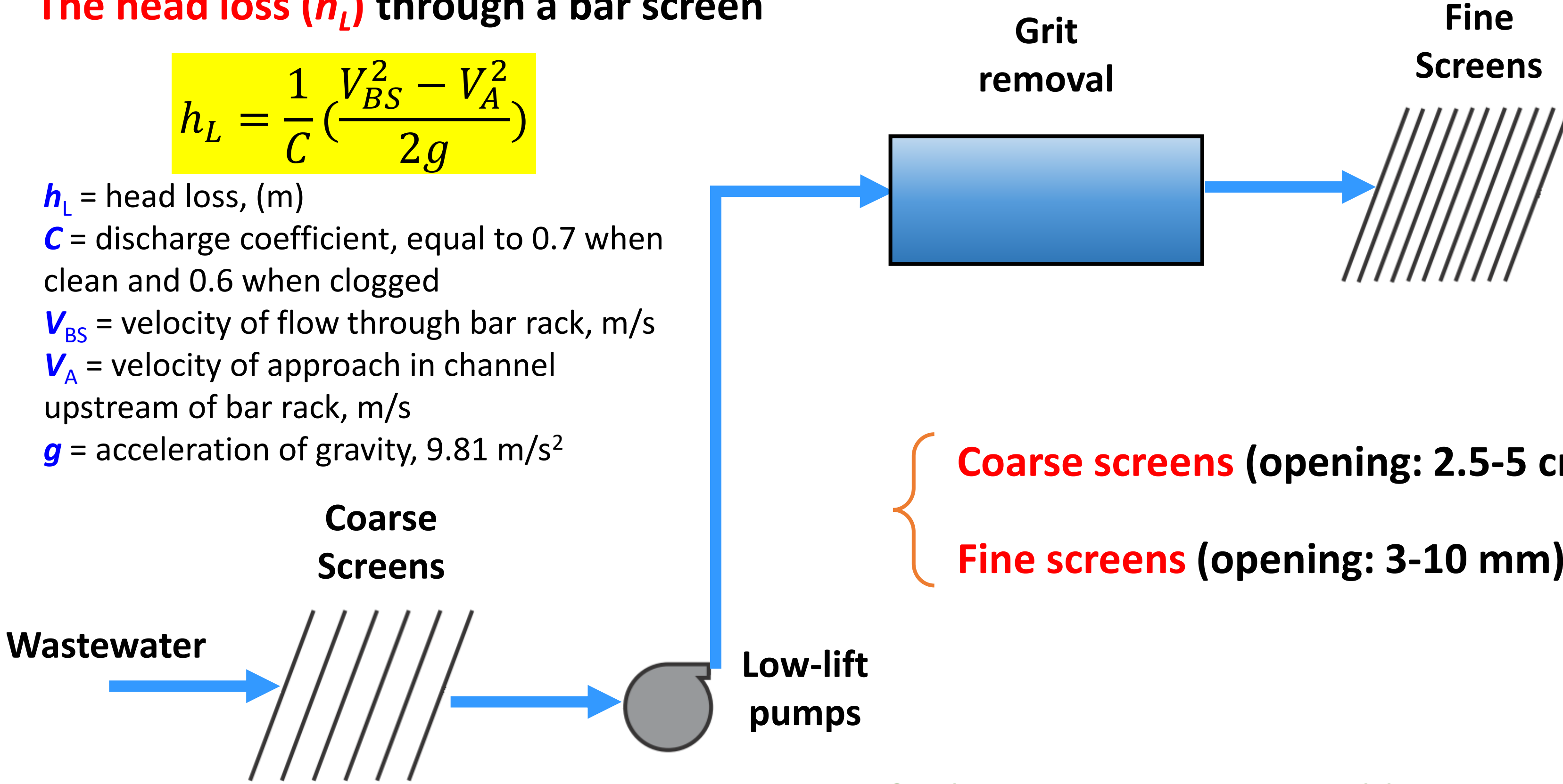
Screens

Screens are necessary for **removing suspended materials** such as large debris, paper, plastics, hair.

The head loss (h_L) through a bar screen

$$h_L = \frac{1}{C}\left(\frac{V_{BS}^2 - V_A^2}{2g}\right)$$

h$_L$ = head loss, (m)
C = discharge coefficient, equal to 0.7 when clean and 0.6 when clogged
V$_{BS}$ = velocity of flow through bar rack, m/s
V$_A$ = velocity of approach in channel upstream of bar rack, m/s
g = acceleration of gravity, 9.81 m/s^2



Coarse screens (opening: 2.5-5 cm)
Fine screens (opening: 3-10 mm)

Design criteria for bar screens can see Table 7.6 in page 340.

Primary treatment

Design of screens

Example 7.3 Bar screen design

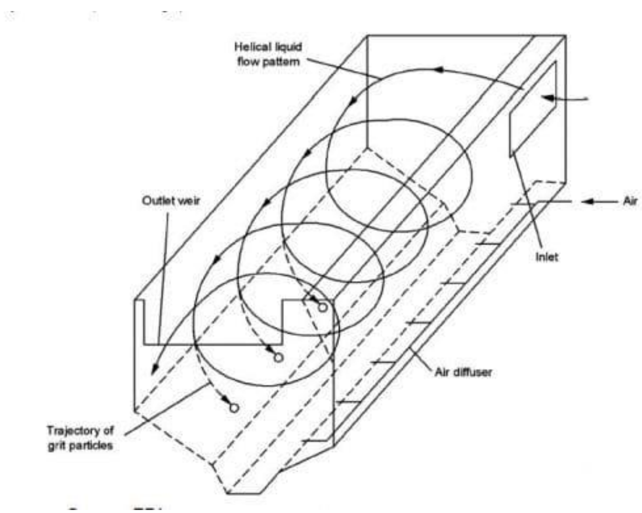
A mechanical bar screen with 1 inch openings and $\frac{5}{8}$ inch bars is installed in a rectangular channel where the approach velocity should not exceed 2.0 ft per second. Estimate:

a) The velocity between the bars.
b) The head loss through the screen, assuming it is clean.

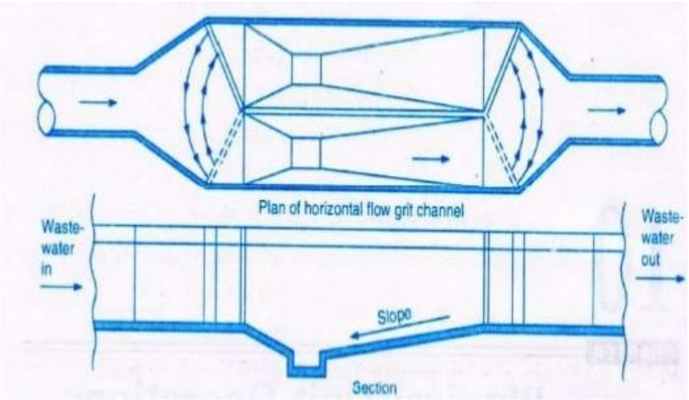
Primary treatment

Grit removal

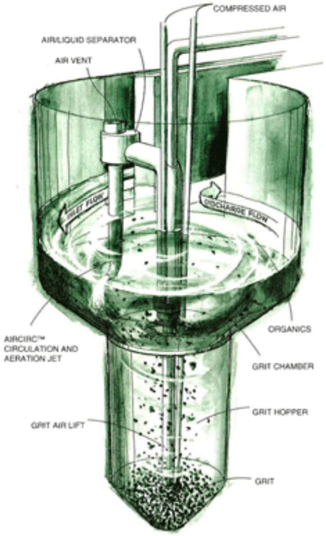
Grit removal is an unit operation that is normally performed during preliminary treatment and follows screening to move sand, silt, small gravel, cinders, coffee grounds, egg shells, and other inert materials that typically have a **specific gravity around 2.65**.



Aerated chamber



Horizontal-flow chamber

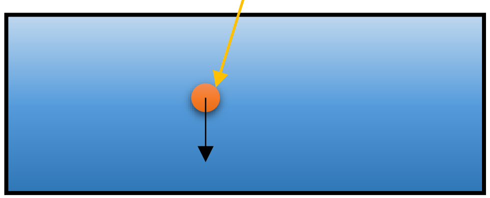


Vortex grit removal systems

Primary treatment

Grit removal

Particle characteristics
Diameter=0.2 mm
Specific gravity = 2.65



Grit removal chamber

Newton's Law

$$v_s = \sqrt{\frac{4g(\rho_p - \rho_w)d_p}{3\lambda\rho_w}}$$

- **In laminar flow**

Stokes' Law

$$v_s = \frac{\rho_p - \rho_w}{18\mu} g {d_p}^2$$

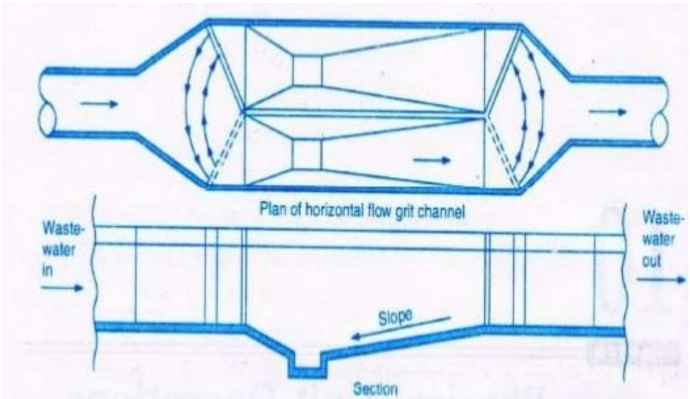
- **In turbulent flow**

$$v_s = \sqrt{3.3g\left(S_p - 1\right)d_p}$$

$\boldsymbol{v_s}$ = settling velocity, m/s
$\boldsymbol{g}$ = acceleration of gravity, m/s^2
$\boldsymbol{\lambda}$ = coefficient of drag (Dimensionless)
$\boldsymbol{\rho_p}$ = mass density of particle, kg/m^3
$\boldsymbol{\rho}$ = mass density of liquid, kg/m^3
$\boldsymbol{S_p}$ = specific gravity =ρ_p/ρ

Primary treatment

Design of Horizontal-flow Grit Chamber



Example 7.4 A horizontal-flow type of grit chamber is designed toremove grit particles with a diameter of 0.2 mm and specific gravity of 2.65. A flow-through velocity of 0.3 m/s will be maintained by a proportioning weir. The average daily wastewater flow is 5,000 m^3/d. The PHF : ADF ratiois 2.0 : 1.0. Determine the channel dimensions for the PHF.

a) Peak hour flow= ratio × ADF = 2 × 5 × 10^3 = 1 × 10^4 m^3/d

b) Cross-sectional area (***A***)= *Q*/*v*= 1 × 10^4 m^3/d ÷ 86400 s/d ÷ 0.3 m/s = 0.39 m^2

c) Cross-sectional area (***A***)= *W* × *H*=*W* × 1.5*W*=0.39 m^2

Assume: *H*=1.5*W*

d) Width=0.51 m; Height=1.5*W*=0.77 m

e) $v_s = \sqrt{3.3g(S_p - 1)d_p} = \sqrt{3.3 \times 9.81 \times (2.65 - 1) \times 2 \times 10^{-4}} = 0.1 \text{ m/s}$

f) $\boldsymbol{\tau}$ = H/v_s = 0.77/0.1=7.7 s

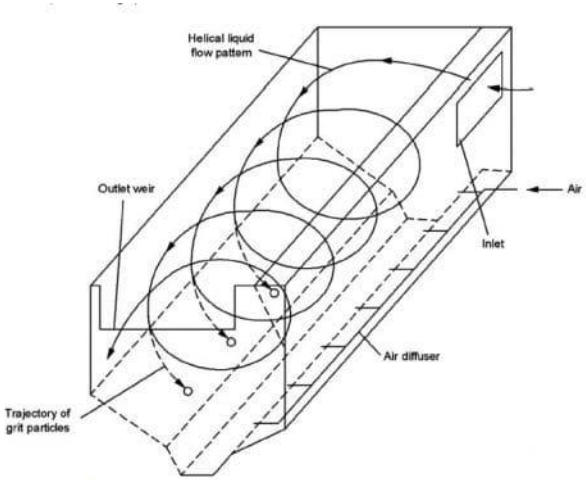
g) L_T= τ × v=7.7 s × 0.3 m/s =2.31 m

h) L=(1+50%)L_T =3.47 m

increasing 50% theoretical length (L_T)

Primary treatment

Design of aerated grit removal chamber



Example 7.4 Design an aerated grit chamber for an average daily wastewater flow rate of 30,000 m^3/d. Assume two grit chambers are operating in parallel. The peak hour flow rate is three times the ADF. Use the design criteria in Table 7.8 to determine the following:

a) The dimensions of each grit chamber.

b) The total air required (m^3/d).

Assuming τ = 3 min

a) Peak hour flow= ratio × ADF = $3 \times 3 \times 10^4 = 9 \times 10^4$ m^3/d = 62.5 m^3/min

b) Volume of each grit chambers (***V***)= $Q \times \tau$ = 1/2 × 62.5 m^3/min × 3 min = 93.75 m^3

c) $V = L \times W \times H = 4W \times W \times W/1.5 = 2.67W^3$ = 93.75 m^3

d) Width=3.3 m; Height=$W/1.5$=2.2 m; Length = $4W$ =13.2 m

e) Air required = 0.5 $m^3/(min \cdot m)$ × 13.2 m × 2=13.2 m^3/min= 792 m^3/h

Assuming:
L/W=4
H/W=1/1.5

Assuming the quantity of air required per unit of length: 0.5 $m^3/(min \cdot m)$

Primary treatment

Primary sedimentation

Primary sedimentation or clarification is a unit operation that involves the separation of settleable solids and the removal of oil, grease, and scum that float to the surface of the wastewater.

Typical removal for *BOD* and *TSS* in primary sedimentation range from 25–40% and 50–70%, respectively.

Rectangular primary clarifier

Circular primary clarifier

Primary treatment

Design of rectangular primary clarifiers

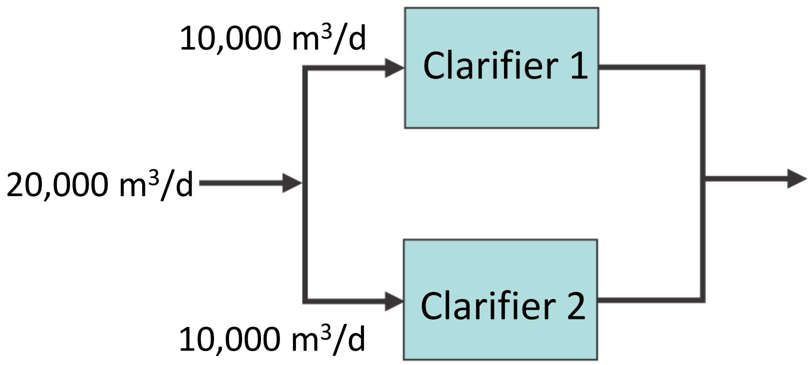
Example 7.6 A municipal WWTP receives an average daily wastewater flow of 20,000 m^3/d. Two, rectangular, primary clarifiers operating in parallel will treat the flow. The peak hour flow anticipated is 2.5 times the average daily flow. Use the design criteria in Tables 7.9 and 7.10, and assume that the effluent weir in each clarifier is eight times the clarifier width. Determine:

a) The **dimension** of each primary clarifier.

b) The **detention time** in each clarifier.

c) The **weir loading rate** ($m^3/(d \cdot m)$) for each clarifier at PHF.

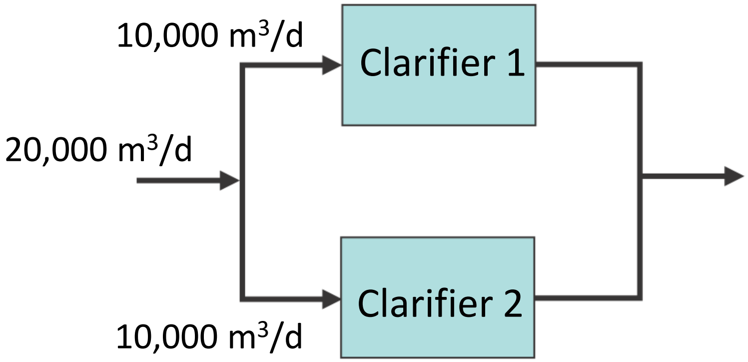
d) The BOD and suspended solids removal efficiencies at ADF.



a) Peak hour flow of each clarifier = ratio $\times$ ADF = $2.5 \times 1 \times 10^4 = 2.5 \times 10^4$ m^3/d

Primary treatment

Design of rectangular primary clarifiers



b) At average daily flow, overflow rate= 33 $m^3/(m^2 \cdot d)$

$A_s = ADF/V_0 = 1 \times 10^4\ m^3/d \div 33\ m^3/(m^2 \cdot d) = 303\ m^2$

c) At peak hour flow, overflow rate= 100 $m^3/(m^2 \cdot d)$

$A_s = PHF/V_0 = 2.5 \times 10^4\ m^3/d \div 100\ m^3/(m^2 \cdot d) = 250\ m^2$

Assuming:
L/W=4

d) $A_s = L \times W = 4W \times W = 303\ m^2$

e) *W*= 8.7 m; *L*= 34.8 m

Freeboard is the distance from the water surface to the top of the wall of the clarifier

f) Side water depth (*SWD*)= 3 m

g) Total height = *SWD*+freeboard = 3+0.5 = 3.5 m

Range: 0.75-2 h

h) $\tau = V/Q = (8.7 \times 34.8 \times 3)/1 \times 10^4\ m^3/d = 0.091\ d = 130.8\ min$

i) $L_{weir} = 8W = 8 \times 8.7\ m = 69.6\ m$

j) $Q_{weir\text{-}peak} = PHF/L_{weir} = 2.5 \times 10^4\ m^3/d \div 69.6\ m = 359\ m^3/(d \cdot m) < 378\ m^3/(d \cdot m)$

k) When ADF= $1 \times 10^4\ m^3/d$, overflow rate=33 m/d and in figure 7.6, the % removal for BOD and suspended solids is 32% and 61%, respectively.

Brief summary

Section 1

- Primary and secondary treatment
- Effluent standards
- Design flows
- Design loadings
- Design of screens
- Design of grit removal chambers
- Design of primary clarifiers

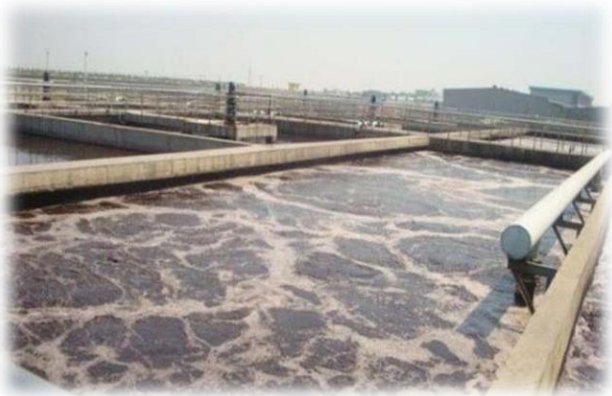
Section 2

Secondary wastewater treatment

Secondary wastewater treatment

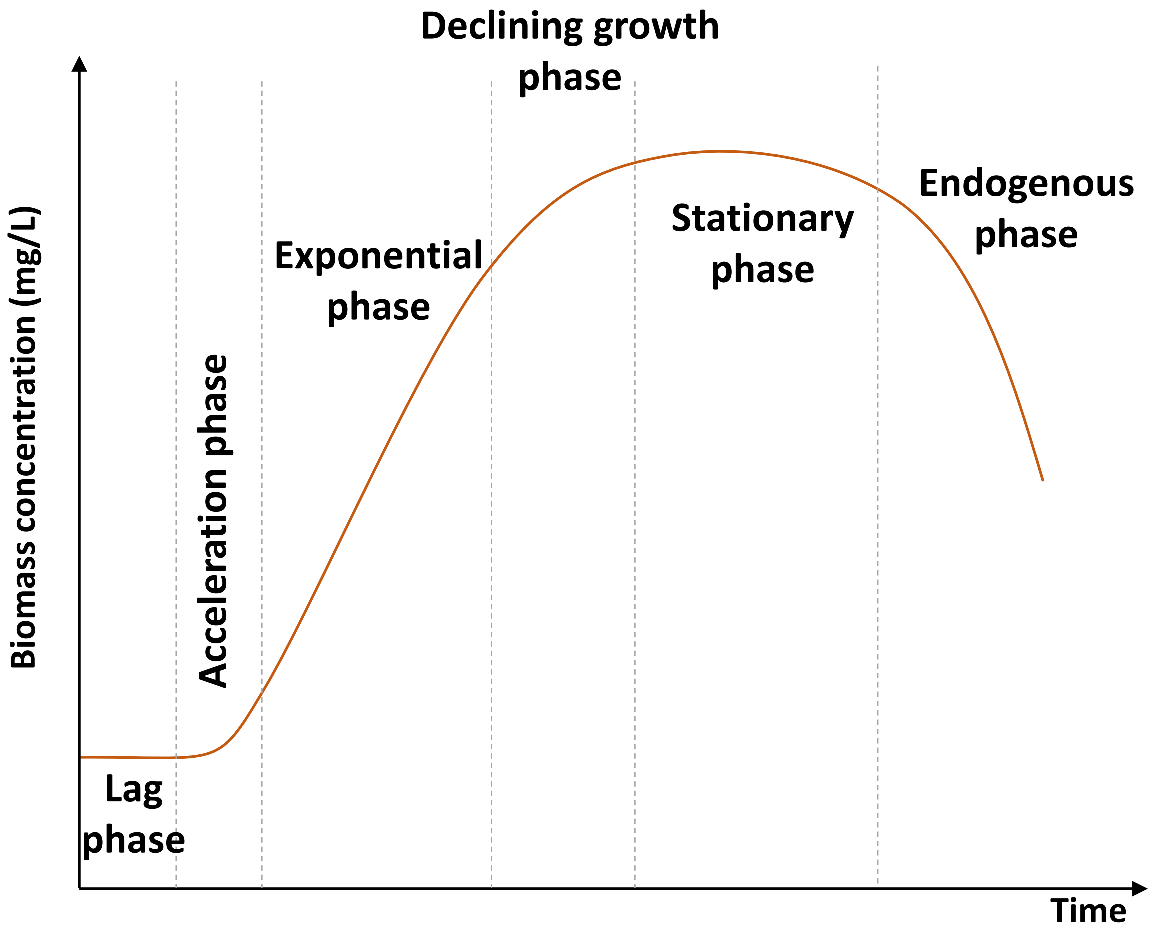
- ***Secondary wastewater treatment*** implies that the major mechanism used for treating the wastewater is a biological process which uses a heterogeneous culture of microorganisms to treat the wastewater.
- ***Suspended growth***: microbes that are suspended in the wastewater.
- ***Attached growth***: microbes that are attached to some type of media.

$$\underbrace{\mathbf{Organics}}_{\text{Organic pollutants}} + \mathbf{O_2} + \mathbf{N} + \mathbf{P} \xrightarrow{\textbf{Microorganisms}} \mathbf{CO_2} + \mathbf{H_2O} + \underbrace{\boldsymbol{C_5H_7O_2NP_{0.16}}}_{\text{New microbial cells}} + \boldsymbol{energy}$$



Secondary wastewater treatment

Microbial growth



Secondary wastewater treatment

Microbial growth in batch reactor

Microorganism growth rate
Mass/(volume·time)

$$\left(\frac{dX}{dt}\right)_G = \mu X$$

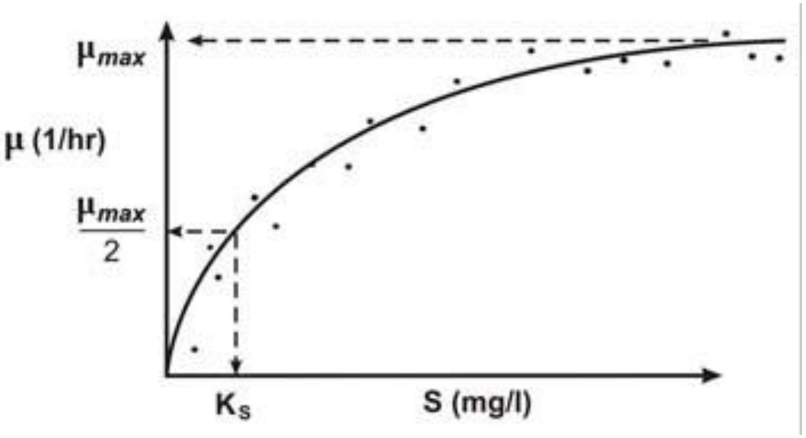
μ **= specific growth rate of microorganism, time**$^{-1}$
X = microorganism concentration, mass/volume

The specific growth rate of a microorganism (***μ***) is associated with a particular species of microorganism.

Monod equation

$$\mu = \mu_{max}\frac{S}{K_s + S}$$

S = growth limiting substrate or nutrient concentration, mass/volume
μ_{max} = maximum specific growth rate, time^{-1}
K_s= half-saturation constant, concentration of limiting substrate or nutrient at which half the maximum specific growth rate occurs, mass/volume.



Secondary wastewater treatment

Microbial growth in continuous flow reactor

Net growth rate
Mass/(volume·time)

$$\left(\frac{dX}{dt}\right)_{NG} = \left(\frac{dX}{dt}\right)_{G} + \left(\frac{dX}{dt}\right)_{ED}$$

Endogenous decay rate
Mass/(volume·time)

$$\mu = \mu_{max}\frac{S}{K_s + S}$$

$$\left(\frac{dX}{dt}\right)_{ED} = -k_d X$$

K_d = endogenous decay rate constant, time^{-1}

$$\left(\frac{dX}{dt}\right)_{NG} = \mu_{max}\frac{XS}{K_s + S} - k_d X$$

Secondary wastewater treatment

Microbial growth in continuous flow reactor

microbial yield coefficient, mass of biomass produced/mass of substrate utilized

$$Y = \frac{(\frac{dX}{dt})_G}{(\frac{dS}{dt})_U}$$

$(\frac{dX}{dt})_G$ → **Microorganism growth rate** Mass/(volume·time)

$(\frac{dS}{dt})_U$ → **substrate utilization rate** Mass/(volume·time)

$$(\frac{dS}{dt})_U = UX$$

***U*= specific substrate utilization rate, time^{-1}**

$$Y = \frac{(\frac{dX}{dt})_G}{(\frac{dS}{dt})_U} = \frac{\mu X}{UX} = \frac{\mu}{U}$$

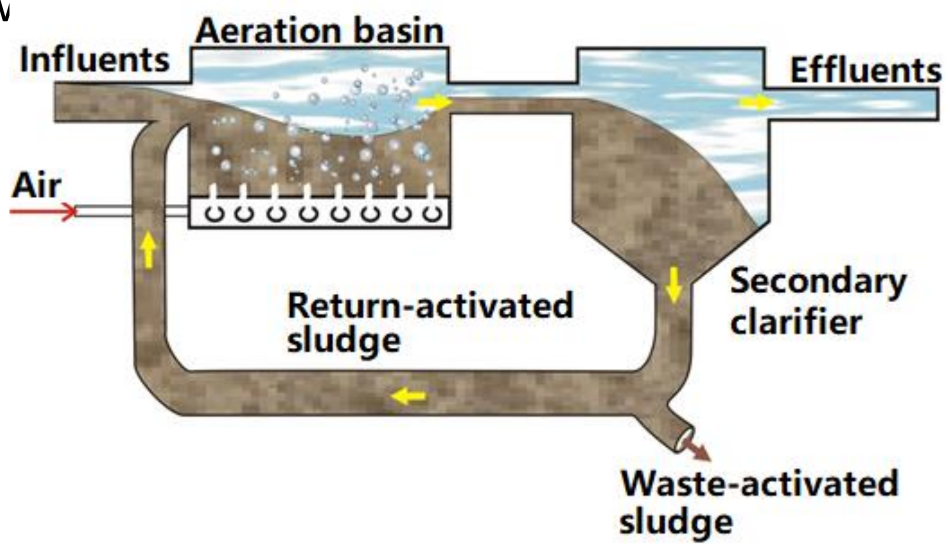
$$(\frac{dX}{dt})_{NG} = (\frac{dX}{dt})_G + (\frac{dX}{dt})_{ED}$$

$$(\frac{dX}{dt})_{NG} = Y(\frac{dS}{dt})_U - k_d X$$

Secondary wastewater treatment

Activated sludge process

- ***The activated sludge process*** is an engineered process wherein oxygen is added to a reactor to speed up the process that naturally occurs in rivers.
- The process is an **aerobic**, suspended growth, biological process used primarily to remove dissolved and colloidal organic matter from w



- Ardern & Lockett (1914) are credited with developing the process in England.

Secondary wastewater treatment

Activated sludge

Activated sludge: after the settling of mixed liquor in the aeration tank, **yellowish-brown flocculent sludge** which plays a major role in purification.

Activated sludge
- **Water** 98%~99%
- **Dry solids** 1%~2% M_a, M_e, M_i, M_{ii}
  - **MLVSS** 70-80% M_a, M_e, M_i
  - **NVSS** 20-30% M_{ii}

MLSS: mixed liquor suspended solids
MLVSS: mixed liquor volatile suspended solids

Secondary wastewater treatment

Activated sludge

Settled volume (SV%)

SV% is the volume (in mL) occupied by the settled sludge after 30 min settling. SV% is determined by filling a 1 L graduated cylinder with a sample of mixed liquor and allowing the solids to settle in a 30-minute period.

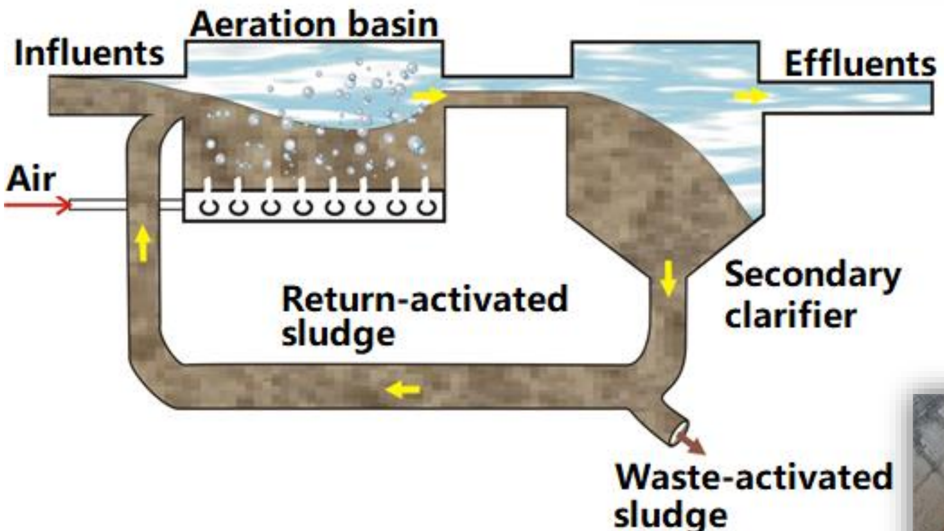
Sludge volume index (SVI)

$$\text{SVI} = \frac{\text{sludge volume after settling (mL/L)}}{\text{Dry weight of TSS in 1 L mixed liquor (g/L)}} = \frac{\text{SV(mL/L)}}{\text{MLSS(g/L)}}$$

- SVI values ranging from **50–150 mL/g** indicate a good settling sludge.
- SVI values greater than 150 mL/g indicate poor settling sludge (**sludge bulking**)

Secondary wastewater treatment

Activated sludge



Sludge concentration in the aeration basin:
MLSS= **1500–3000** mg/L

TSS concentration of WAS :
MLSS= **5000–10000 mg/L**

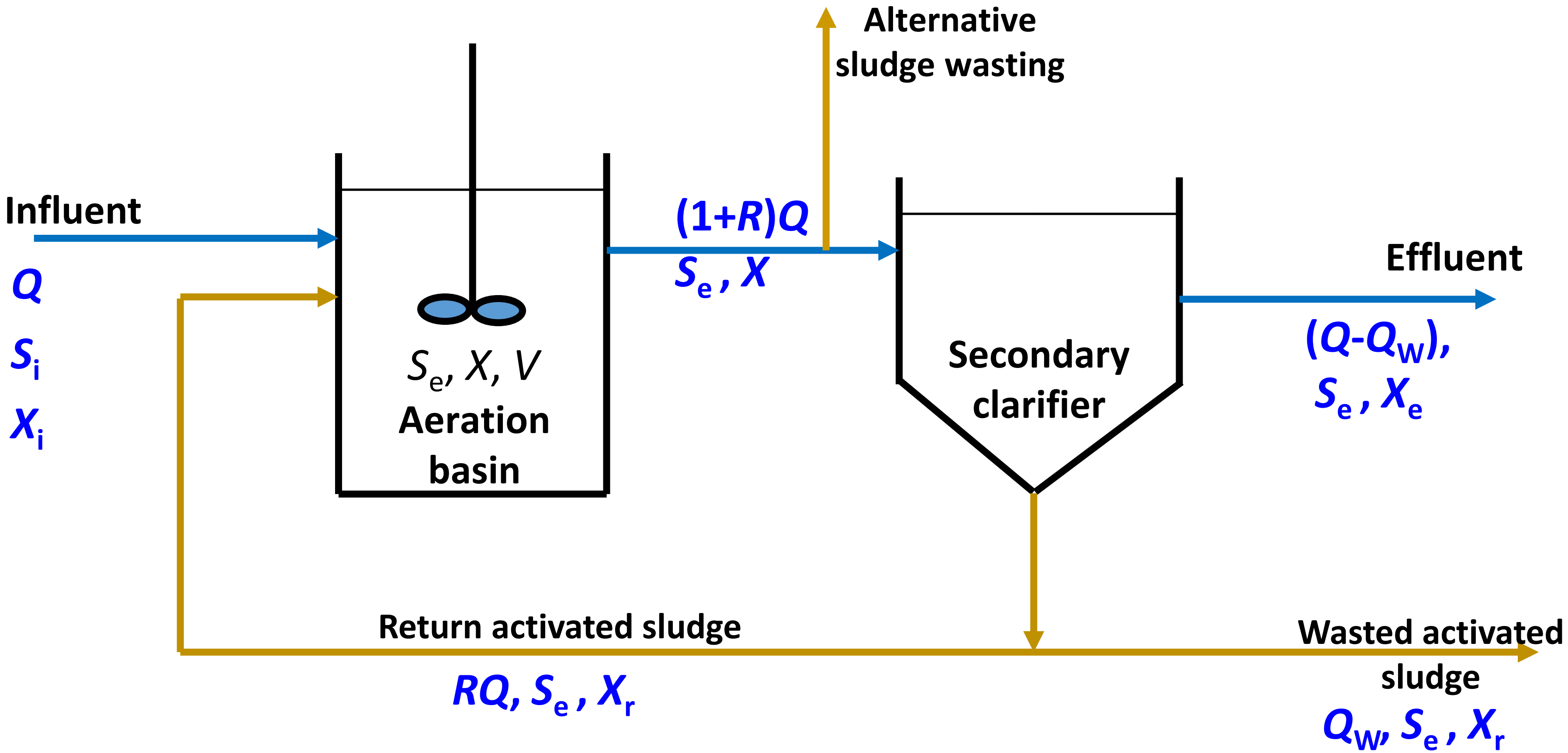
Secondary wastewater treatment

Design and operational parameters

- Mean Cell Residence Time (*MCRT*)
- Hydraulic Retention(Detention) Time (*HRT*)
- Food-to-Microorganism Ratio (*F/M*)
- Specific Substrate Utilization Rate (*U*)
- Efficiency of Treatment (*E*)

Secondary wastewater treatment

Design and operational parameters



Secondary wastewater treatment

Mean Cell Residence Time (*MCRT*)

MCRT (**ϑ_c**), also called solids retention time (***SRT***) or sludge age, is the total biomass in the system divided by the biomass wasted from the system.

$$\boldsymbol{\theta_c} = \frac{XV}{(Q - Q_W)X_e + Q_W X_r}$$

θ_c = mean cell residence time, days
X = active biomass concentration in aeration basin measured as MLSS or MLVSS concentration, mg/L
V = volume of the aeration basin, m^3
X_e = secondary effluent TSS or VSS concentration, mg/L
X_r = TSS or VSS concentration in return activated sludge, mg/L
Q = influent wastewater flow rate, m^3/d
Q_w = sludge wastage flow rate, m^3/d.

- MCRT generally varies from **5–30 days** and determines the overall substrate removal efficiency of the process.
- The longer the MCRT, the lower the effluent soluble substrate concentration
- an accumulation of microbial end-products and the release of secondary substrate products occurs at long MCRTs

Hydraulic Retention Time (*HRT*)

Hydraulic retention (detention) time (HRT, τ), is the time that the wastewater resides in a particular unit operation or process.

$$\boldsymbol{\tau} = \frac{V}{Q}$$

$\boldsymbol{\tau}$ = hydraulic retention time, d or h
V = volume of unit operation or process, m^3
Q = influent flow to unit operation or process excluding recycles, m^3/d

MCRT (5-30 d) is much longer than the hydraulic detention time (τ) which usually in 6-20 h.

Food-to-Microorganism Ratio (*F/M*)

Food-to-Microorganism Ratio (F/M), also called sludge loading, is used to determine the volume of the aeration basin by assuming values for the *F/M* ratio, *X*, *Q*, and S_i.

$$F/M = \frac{QS_i}{XV}$$

F/M = food-to-microorganism ratio, d^{-1}
S_i = influent substrate concentration prior to mixing with sludge recycle expressed as BOD or COD, mg/L
V = volume of unit operation or process, m^3
Q = wastewater flow rate prior to mixing with the RAS flow, m^3/d

Secondary wastewater treatment

Specific Substrate Utilization Rate (*U*)

$$\boldsymbol{U} = \frac{Q(S_i - S_e)}{XV} \quad \Longleftrightarrow \quad \boldsymbol{F/M} = \frac{QS_i}{XV}$$

U = specific substrate utilization rate, d^{-1}

$\boldsymbol{S_e}$ = effluent soluble substrate concentration expressed as soluble BOD (SBOD) or soluble COD (SCOD), mg/L

Efficiency of Treatment (*E*)

$$\boldsymbol{E} = \frac{(C_i - C_e)}{C_i} \times 100\%$$

E = efficiency of treatment expressed as a percentage

$\boldsymbol{C_i}$ = concentration of parameter in the influent to process, mg/L

$\boldsymbol{C_e}$ =concentration of parameter in the effluent from process, mg/L.

Example 7.7 Calculating activated sludge operating parameters

A complete-mix activated sludge process treats 20000 m^3/d of wastewater containing 225 mg/L BOD_5 to 20 mg/L BOD_5. The volume of the aeration basin is 3150 m^3 and the MLSS concentration is 2500 mg/L. Determine the following:

a) Detention time in the aeration basin.

b) F/M ratio.

c) Specific substrate utilization rate.

d) Substrate removal efficiency.

a) $\tau = V/Q$ = 3150 m^3/20000 m^3/d = 0.158 d = **3.8 h**

b) $F/M = QS_i/XV$ = 20000 m^3/d × 225 mg/L ÷ (3150 m^3 × 2500 mg/L)

= **0.57 $mgBOD_5/(mgTSS \cdot d^{-1})$**

a) $U = Q(S_i-S_e)/XV$ = 20000 m^3/d × (225 mg/L-20 mg/L) ÷ (3150 m^3 × 2500 mg/L)

= **0.52 $mgBOD_5/(mgTSS \cdot d^{-1})$**

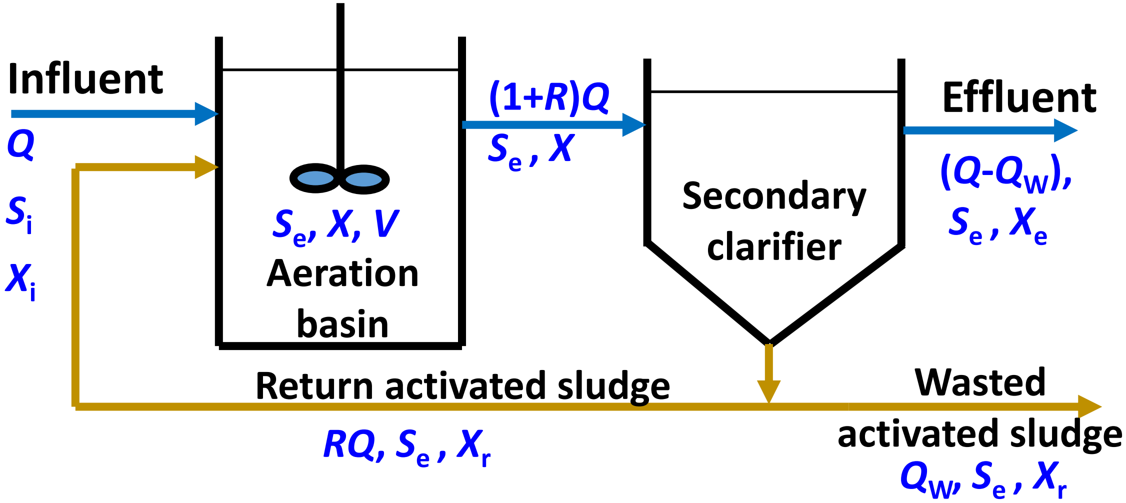
a) Substrate removal efficiency = $(S_i-S_e)/S_i$ = (225 mg/L-20 mg/L)/225 mg/L × 100%

=91.1%

Secondary wastewater treatment

Biochemical kinetics of complete-mix activated sludge (CMAS) systems

(Design of complete-mix activated sludge system)

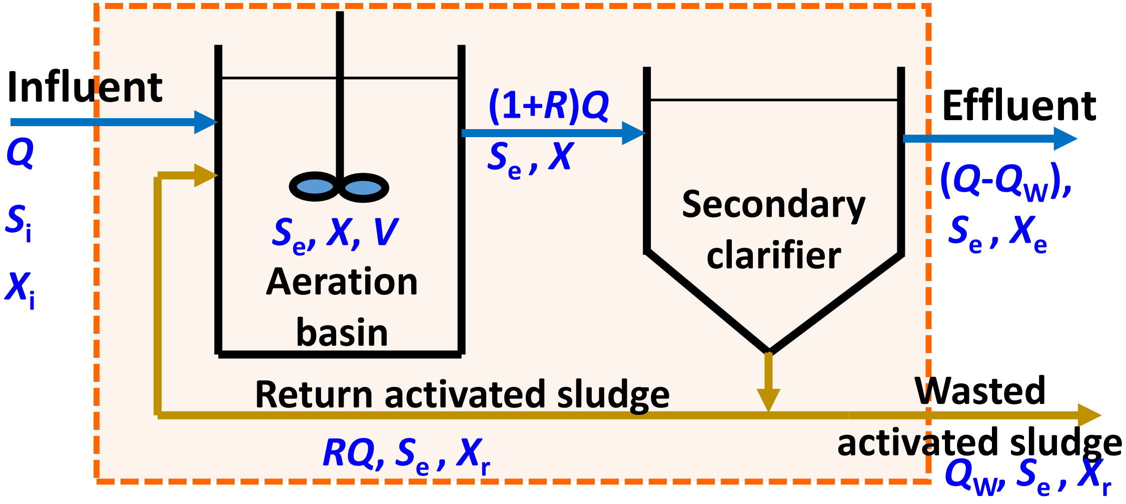


Assumptions

1. Steady-state conditions;
2. Complete mixing is achieved in the aeration basin;
3. Biological activity only occurs in the aeration basin, not in secondary clarifier;
4. The concentration of microorganisms in the influent wastewater is negligible;
5. The mean cell residence time is based only on the biomass in the aeration;
6. No sludge accumulation occurs in the secondary clarifier;
7. The substrate concentration is measured as SBOD or SCOD;
8. Sludge is purposely wasted from the sludge return line.

Secondary wastewater treatment

Design of CMAS system-Net Growth Rate Equation



Perform a materials balance on biomass (X) around the entire system

$$[accumulation] = [inputs] - [outputs] + [reaction]$$

$$V\left(\frac{dX}{dt}\right)_{accu} = QX_i - (Q - Q_W)X_e - Q_W X_r + \left(\frac{dX}{dt}\right)_{NG} V$$

Secondary wastewater treatment

Design of CMAS system-Net Growth Rate Equation

$$[accumulation] = [inputs] - [outputs] + [reaction]$$

$$V(\frac{dX}{dt})_{accu} = QX_i - (Q - Q_W)X_e - Q_W X_r + (\frac{dX}{dt})_{NG} V$$

In steady condition, and X_i=0

$$0 = 0 - (Q - Q_W)X_e - Q_W X_r + (\frac{dX}{dt})_{NG} V$$

$$(\frac{dX}{dt})_{NG} = \frac{(Q - Q_W)X_e + Q_W X_r}{V}$$

$$\frac{1}{X}(\frac{dX}{dt})_{NG} = \frac{(Q - Q_W)X_e + Q_W X_r}{XV} = \frac{1}{\theta_c}$$

Remember

$$(\frac{dX}{dt})_{NG} = Y(\frac{dS}{dt})_U - k_d X$$

$$\frac{1}{\theta_c} = Y\frac{1}{X}(\frac{dS}{dt})_U - k_d = \mu - k_d$$

$$\boxed{\boldsymbol{\frac{1}{\theta_c} = \mu - k_d}}$$

Secondary wastewater treatment

Design of CMAS system-Effluent Soluble Substrate Equations

Substrate utilization equation

$$\boxed{\left(\frac{dS}{dt}\right)_U = \frac{kXS_e}{K_s + S_e}}$$

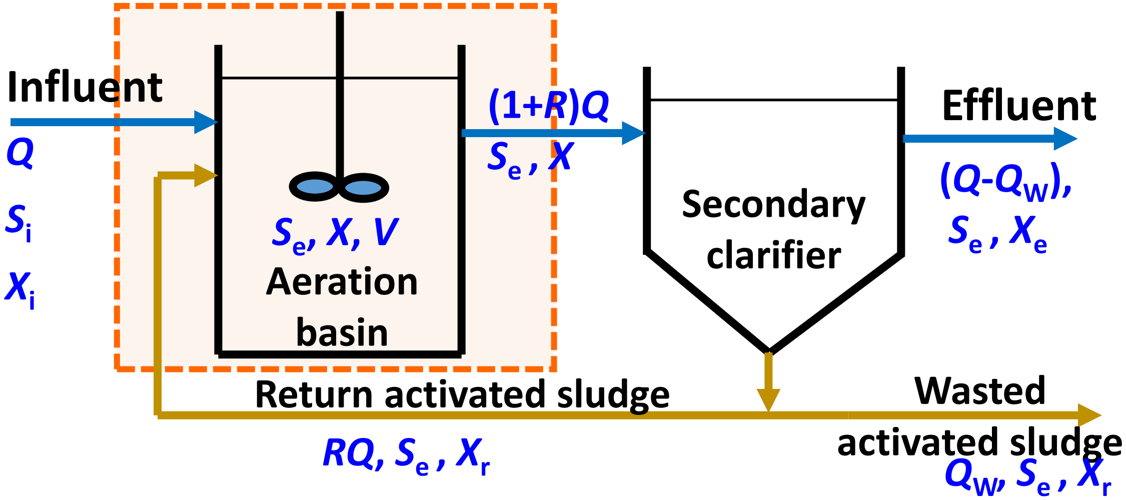
k = maximum specific substrate utilization rate, d^{-1}
$\boldsymbol{K_s}$ = half-velocity constant, substrate concentration at one-half the maximum specific substrate utilization rate, mg/L

$$\frac{1}{\theta_c} = Y\frac{1}{X}\left(\frac{dS}{dt}\right)_U - k_d \longrightarrow \frac{1}{\theta_c} = Y\frac{1}{X}\frac{kXS_e}{K_s + S_e} - k_d$$

$$\boxed{\boldsymbol{S_e} = \frac{K_s(1 + k_d\boldsymbol{\theta_c})}{\boldsymbol{\theta_c}(Yk - k_d)}} \longleftarrow \frac{1}{\theta_c} = Y\frac{kS_e}{K_s + S_e} - k_d$$

Secondary wastewater treatment

Design of CMAS system-Aeration Basin Biomass Concentration Equation



Perform a materials balance on substrate (S) around the aeration basin.

$$[accumulation] = [inputs] - [outputs] + [reaction]$$

$$V\left(\frac{dS}{dt}\right)_{accu} = QS_i - RQS_e - Q(1+R)S_e - \left(\frac{dS}{dt}\right)_U V$$

Design of CMAS system-Aeration Basin Biomass Concentration Equation

$$[accumulation] = [inputs] - [outputs] + [reaction]$$

$$V(\frac{dS}{dt})_{accu} = QS_i + RQS_e - Q(1+R)S_e - (\frac{dS}{dt})_U V$$

In steady condition,

$$0 = QS_i + RQS_e - Q(1+R)S_e - (\frac{dS}{dt})_U V$$

$$(\frac{dS}{dt})_U = \frac{Q(S_i - S_e)}{V}$$

$$\frac{1}{\theta_c} = Y\frac{1}{X}(\frac{dS}{dt})_U - k_d$$

$$X = \frac{YQ(S_i - S_e)}{V(\frac{1}{\theta_c} + k_d)}$$

$$V = \frac{YQ(S_i - S_e)}{X(\frac{1}{\theta_c} + k_d)}$$

Secondary wastewater treatment

Design of CMAS system-Sludge Production Equation

$$Y_{obs} = \frac{(dX/dt)_{NG}}{(dS/dt)_U}$$

$$\frac{1}{\theta_c} = Y\frac{1}{X}(\frac{dS}{dt})_U - k_d$$

$$Y_{obs} = \frac{Y}{1+\theta_c k_d}$$

Quantity of excess biomass produced on a dry weight basis, kgTSS/d

$$\Delta \boldsymbol{X} = Y_{obs}Q(S_i - S_e)$$

$$\Delta \boldsymbol{X} = \frac{YQ(S_i - S_e)}{1+\theta_c k_d}$$

Secondary wastewater treatment

Design of CMAS system-Oxygen Requirements

$$O_2 = Q(S_i - S_e)(1 - 1.42Y) + 1.42k_d XV + NOD$$

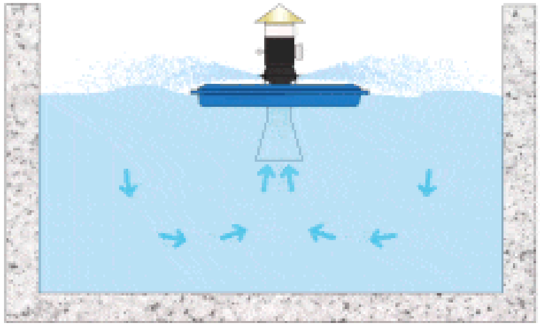
O_2 = total quantity of oxygen required to meet process, kg/d
NOD = nitrogenous oxygen demand that occurs during nitification, kg/d

$$NOD = 4.57Q(TKN_o)$$

TKN_o = total Kjeldahl nitrogen concentration entering the aeration basin exluding RAS flow, mg/L
4.57 =quantity of oxygen required to oxidize NH_3 into NO_3^-, $kgO_2/kgNH_3$-N

A minimum of **2.0 mg/L** of dissolved oxygen (DO) is maintained in the aeration basin to meet process oxygen requirements.

Diffused aeration



Mechanical aerator

Secondary wastewater treatment

Design of CMAS system

Background

- Flow rate (***Q***)
- BOD_5 Concentration in influent (S_i)
- BOD_5 Concentration in effluent (S_e)
- Temperature of process (T)

Biokinetics parameters

- Microbial yield coefficient (Y)
- Specific growth rate of microorganism (μ)
- Endogenous decay rate constant (k_d)
- Half-saturation constant (k_s)

Designing parameters

- Mean Cell Residence Time (θ_c)
- Biomass Concentration (***TSS***)
- Volume of aeration basin (V)
- Oxygen demand (O_2)
- Excess sludge production (ΔX)

Example 7.9 Activated sludge design without nitrification

A complete-mix activated sludge process (CMAS) is to be designed to treat 25000 m^3/d of domestic wastewater having a BOD_5 of 250 mg/L. The NPDES permit requires that the effluent BOD_5 and TSS concentrations should be 20 mg/L or less on an annual average basis (effluent soluble BOD_5 should be less than 5 mg/L). The following biokinetic coefficients obtained at 20°C will be used in designing the process: Y = 0.6 mg VSS/mg BOD_5, k=5 d^{-1}, K_s=60 mg/L BOD_5, and k_d = 0.06 d^{-1}. Assume that the MLVSS concentration in the aeration basin is maintained at 3000 mg/L and the VSS : TSS ratio is 0.80. Determine the following:

a) Mean cell residence time necessary to meet the NPDES permit in 20°C.
b) Volume of the aeration basin in m^3.
c) Oxygen requirements assuming no nitrification in 20 °C.
d) The quantity of excess biomass produced in terms of TSS.

Secondary wastewater treatment

a) Mean cell residence time necessary to meet the NPDES permit in 20°C.

$$\frac{1}{\theta_c} = Y\frac{1}{X}(\frac{dS}{dt})_U - k_d = Y\frac{1}{X}\frac{kXS_e}{K_s + S_e} - k_d$$

$$\frac{1}{\theta_c} = 0.6\ \frac{\mathrm{mgVSS}}{\mathrm{mg\ BOD_5}}\frac{5\ \mathrm{d^{-1}} \times 5\ \mathrm{mg/L}}{60\ \mathrm{mg/L} + 5\ \mathrm{mg/L}} - 0.06\ \mathrm{d^{-1}} = 0.17\mathrm{d^{-1}}$$

$$\boxed{\theta_c = 5.9\ \mathrm{d}}$$

b) Volume of the aeration basin in m³.

$$V = \frac{YQ(S_i - S_e)}{X(\frac{1}{\theta_c} + k_d)} = \frac{0.6\ \frac{\mathrm{mgVSS}}{\mathrm{mg\ BOD_5}} \times 25000\ \mathrm{m^3/d} \times (250\ \mathrm{mg/L} - 5\ \mathrm{mg/L})}{3000\ \mathrm{mgVSS/L} \times (0.17\mathrm{d^{-1}} + 0.06\ \mathrm{d^{-1}})}$$

$$\boxed{V = 5326\ \mathrm{m^3}}$$

Secondary wastewater treatment

c) Oxygen requirements assuming no nitrification in 20 °C.

$$O_2 = Q(S_i - S_e)(1 - 1.42Y) + 1.42k_d XV + NOD$$

$$S_i - S_e = 250\ \text{mg/L} - 5\ \text{mg/L} = 245\ \text{mg/L} = 0.245\ \text{kg/m}^3$$

$$X = 3000\ \text{mgVSS/L} = 3\ \text{kgVSS/m}^3$$

$$O_2 = 25000\frac{\text{m}^3}{\text{d}} \times 0.245\ \text{kg/m}^3 \times \left(1 - 1.42 \times 0.6\ \frac{\text{mgVSS}}{\text{mg BOD}_5}\right)$$
$$+1.42 \times 0.06\ \text{d}^{-1} \times 3\ \text{kgVSS/m}^3 \times 5326\ \text{m}^3 + 0$$

$$\boxed{O_2 = 2268\ \text{kg/d}}$$

d) The quantity of excess biomass produced in terms of TSS.

$$\Delta \boldsymbol{X} = \frac{YQ(S_i - S_e)}{1 + \theta_c k_d} = \frac{25000\frac{\text{m}^3}{\text{d}} \times 0.245\ \text{kg/m}^3 \times 0.6\ \frac{\text{mgVSS}}{\text{mg BOD}_5}}{1 + 5.9\ \text{d} \times 0.06\ \text{d}^{-1}}$$

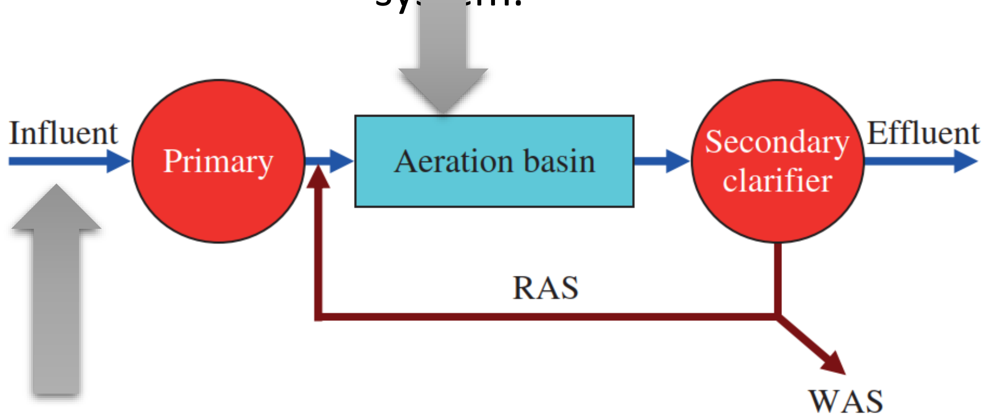
$$\boxed{\Delta \boldsymbol{X} = 2714\ \text{kgVSS/d} = 3393\ \text{kgTSS/d}}$$

Secondary wastewater treatment

Activated sludge modifications

A. Conventional activated sludge process

The oxygen demand of the wastewater at the front of the aeration tank exceeds the capacity of the aeration system.

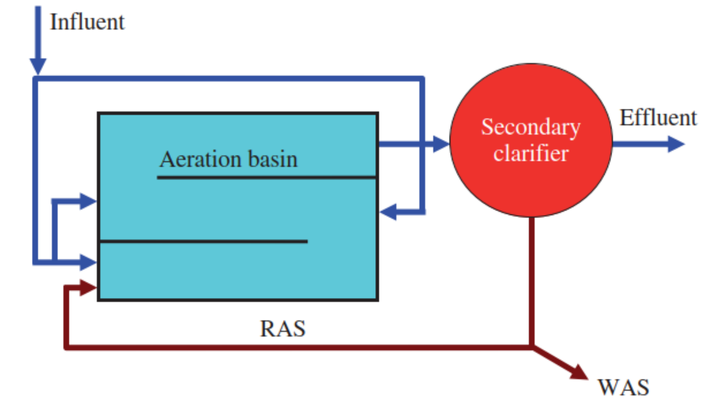


Shock loadings due to toxic and/or high-strength waste entering the aeration tank often result in process failure.

Secondary wastewater treatment

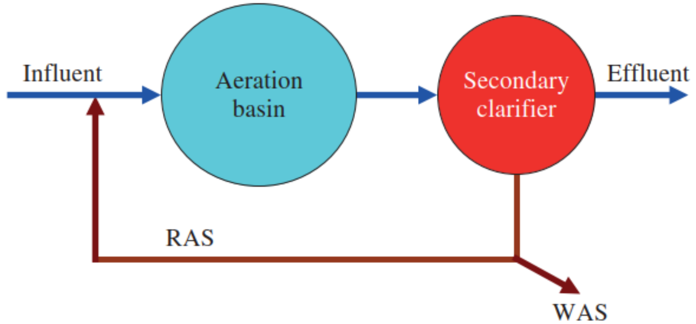
Activated sludge modifications

B. Step feed activated sludge process



Equalizing the oxygen demand incurred from the wastewater.

C. Complete-mix activated sludge process



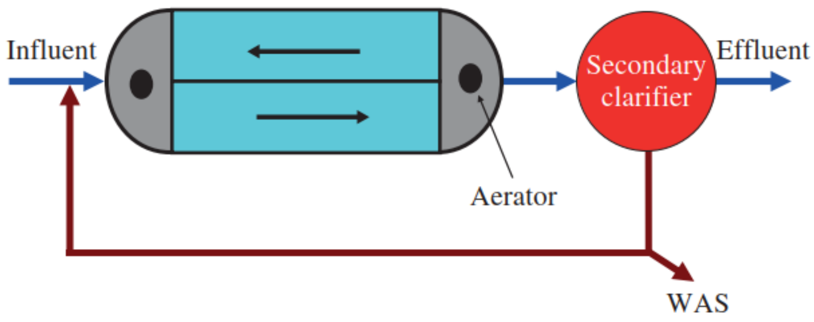
This precludes any problems with shock loadings due to toxic and high-strength wastes entering the reactor.

Secondary wastewater treatment

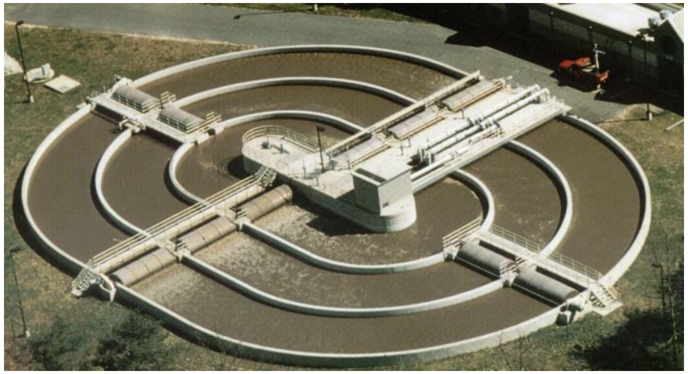
Activated sludge modifications

D. **Extended Aeration activated sludge process**

E. **Oxidation Ditch**



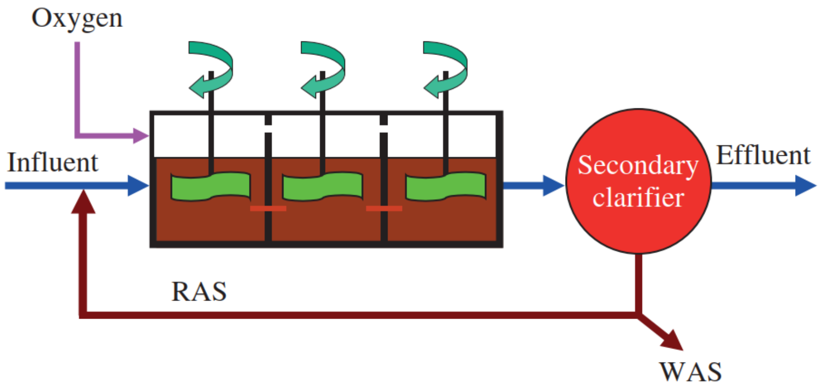
Excess sludge production is very low



Secondary wastewater treatment

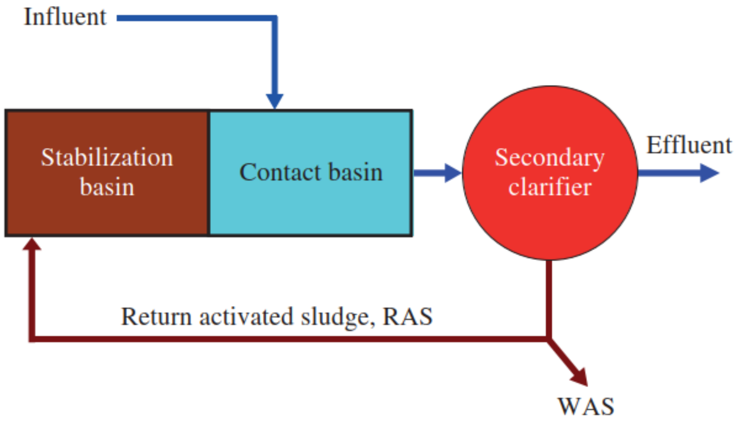
Activated sludge modifications

F. High-Purity Oxygen Activated Sludge Process



- Higher strength wastewaters
- Smaller reactor volumes

G. Contact Stabilization Activated Sludge Process

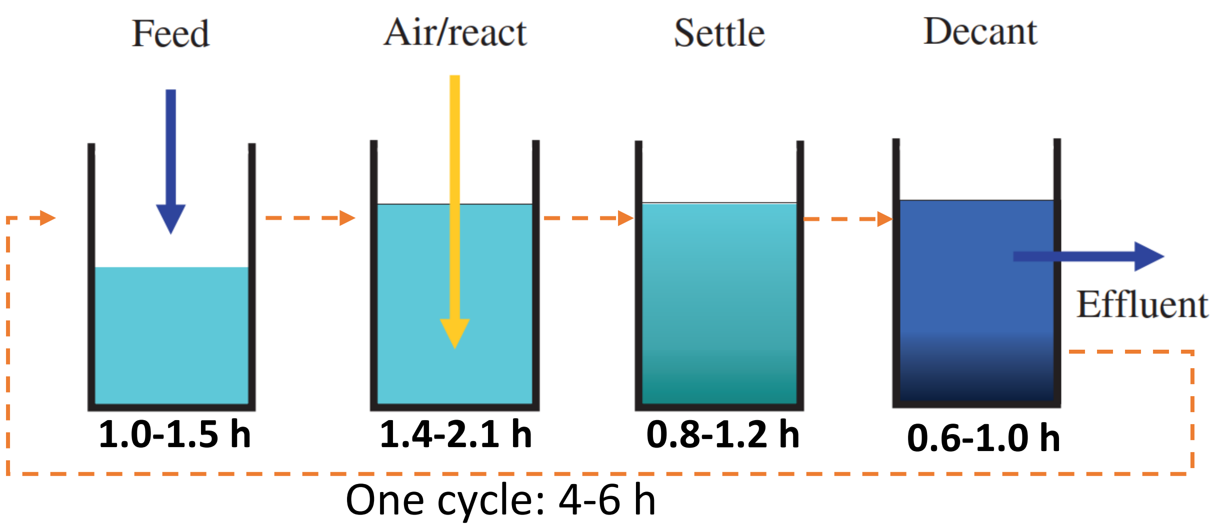


- τ in contact basin: 0.5-1.0 hour
- τ in stabilization basin: 3-6 hour

Secondary wastewater treatment

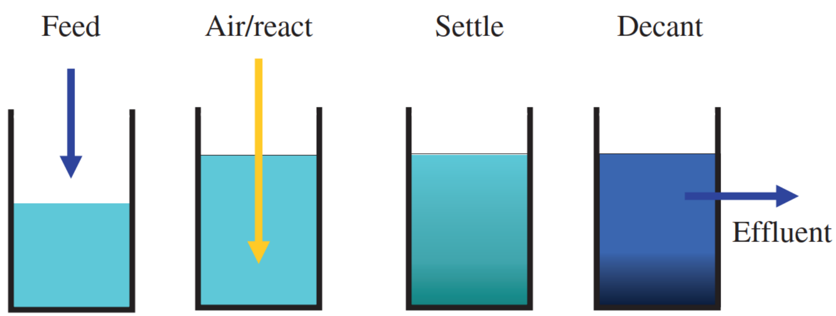
Sequencing batch reactor (SBR)

Sequencing batch reactors (SBRs) is a single tank or reactor which is used to accomplish biological treatment of the wastewater and solids/liquid separation.



Secondary wastewater treatment

Sequencing batch reactor (SBR)



Decanter

Advantages

I. A single basin is used to grow the microorganisms and separate the biomass from the wastewater without the need of a secondary clarifier.
II. The time needed to achieve a specified level of treatment can be varied to accommodate changes in influent wastewater characteristics.
III. It is possible to produce anaerobic, anoxic, and aerobic phases of operation with an SBR, so that biological nitrogen and phosphorus removal can be accomplished.

Disadvantages

I. SBRs are usually limited to wastewater flows that are 18,900 m3/d or less.
II. A minimum of two, and preferably three, SBRs should be used so that it is possible to treat incoming wastewater flows.
III. post-equalization (holding tank) is required.

Secondary wastewater treatment

Brief summary

Secondary wastewater treatment

- Microbial growth in batch reactor
- Activated sludge process
- Design and operational parameters
- Design of complete-mix activated sludge system
- Activated sludge process modifications
- SBR

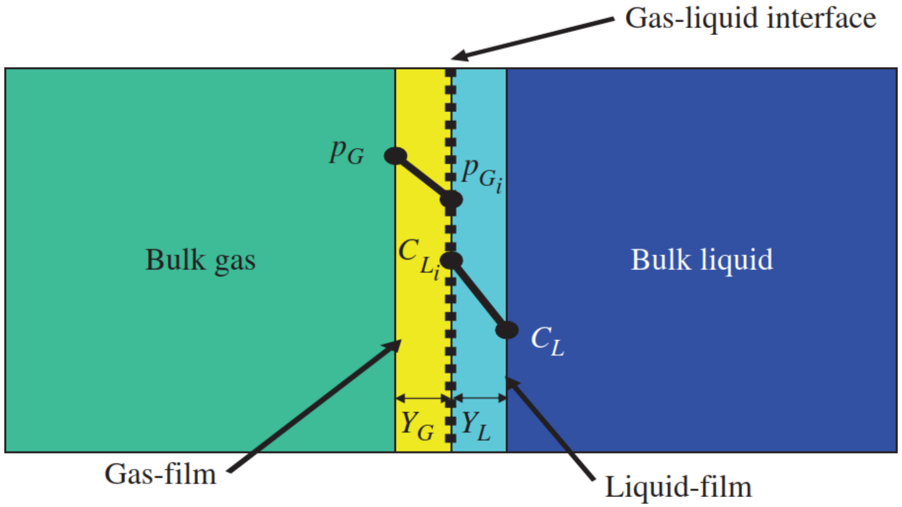
Section 2

Oxygen transfer and mixing

Oxygen transfer and mixing

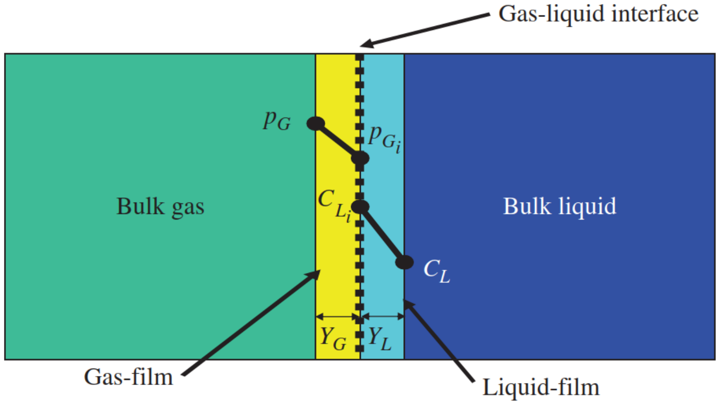
Two-film theory

1. Gas molecules are transported to the outer face of the **gas-film** by mixing and diffusion.
2. Then they diffuse across the gas-film to the gas-liquid **interface**, where the gas molecules dissolve in the liquid-film.
3. Next, the dissolved gas diffuses through the **liquid-film** to the boundary between the film and bulk liquid phase.
4. Finally, the dissolved gas molecules are transported throughout the bulk liquid phase by **mixing**.



Oxygen transfer and mixing

Derivation of mass transfer equation



At steady-state conditions, The rate of transfer of gas A diffusing through a stagnant gas-film and stagnant liquid-film:

$$\frac{dm_A}{dt} = K_G A(P_G - P_i) = K_L A(C_{Li} - C_L)$$

The rate of diffusion or mass transfer of these gases is controlled by the resistance to transfer from the liquid-film, meaning (***$K_G \gg K_L$***).

$$\frac{dm_A}{dt} = K_L A(C_{Li} - C_L)$$

$$\frac{dm_A}{dt} = K_L A(C_s - C_L)$$

The saturation concentration

$$\boxed{\frac{dC}{dt} = K_L a(C_s - C_L)}$$

$$K_L a = K_L \frac{A}{V}$$

dC/dt= overall oxygen transfer rate, mass/(volume · time)
$K_L a$ = overall oxygen transfer coefficient, time^{-1}
A/V= a interfacial surface area, L^2
C_s = dissolved oxygen saturation concentration for a given temperature and pressure, mass/volume.
C_L = actual dissolved oxygen concentration in the aeration basin or tank, mass/volume.

Oxygen transfer and mixing

Types of aeration systems

Mechanical aerator

Diffused aeration system

Oxygen transfer and mixing

Derivation of oxygen transfer equation

The actual oxygen transfer rate (*AOTR*) represents the quantity of oxygen that the aerator or aeration system can provide at process conditions.

The standard oxygen transfer rate (*SOTR*) is the rate at which the aeration device transfers oxygen in clean tap water at standard conditions of 20∘C and 1 atmosphere of pressure.

- the constituents in the wastewater that affect the transfer rate (***α***)
- the constituents in the wastewater that affect dissolved oxygen saturation concentration (***β***)
- Temperature also affects the $K_L a$ value and the Cs concentration (***θ***).

$$\frac{dC}{dt} = K_L a(C_s - C_L)$$

$$\left(\frac{dC}{dt}\right)_{\text{actual}} = \alpha (K_L a)_{20^\circ\text{C}} (\beta C_s - C_t)(\theta)^{(T-20^\circ\text{C})}$$

Oxygen transfer and mixing

Derivation of oxygen transfer equation

$$\left(\frac{dC}{dt}\right)_{\mathrm{stan}} = (K_L a)_{20℃}(C_s)_{20℃}$$

$$\left(\frac{dC}{dt}\right)_{\mathrm{actual}} = \alpha (K_L a)_{20℃}(\beta C_s - C_t)(\theta)^{(T-20℃)}$$

$$\frac{\left(\frac{dC}{dt}\right)_{\mathrm{actual}}}{\left(\frac{dC}{dt}\right)_{\mathrm{stan}}} = \frac{\alpha (K_L a)_{20℃}(\beta C_s - C_t)(\theta)^{(T-20℃)}}{(K_L a)_{20℃}(C_s)_{20℃}}$$

$$\frac{\left(\frac{dC}{dt}\right)_{\mathrm{actual}}}{\left(\frac{dC}{dt}\right)_{\mathrm{stan}}} = \frac{\alpha\, (\beta C_s - C_t)(\theta)^{(T-20℃)}}{(C_s)_{20℃}}$$

DO saturation concentration at 20℃ and 1 atmosphere of pressure is approximately **9.17 mg/L**

$$\left(\frac{dC}{dt}\right)_{\mathrm{actual}} = \left(\frac{dC}{dt}\right)_{\mathrm{stan}} \frac{\alpha\, (\beta C_s - C_t)(\theta)^{(T-20℃)}}{9.17}$$

Oxygen transfer and mixing

Derivation of oxygen transfer equation

$$\left(\frac{dC}{dt}\right)_{\text{actual}} = \left(\frac{dC}{dt}\right)_{\text{stan}} \frac{\alpha\,(\beta C_s - C_t)(\theta)^{(T-20℃)}}{9.17}$$

$$V\left(\frac{dC}{dt}\right)_{\text{actual}} = V\left(\frac{dC}{dt}\right)_{\text{stan}} \frac{\alpha\,(\beta C_s - C_t)(\theta)^{(T-20℃)}}{9.17}$$

Unit: **kg/d**

$$ATOR = STOR\,\frac{\alpha\,(\beta C_s - C_t)(\theta)^{(T-20℃)}}{9.17}$$

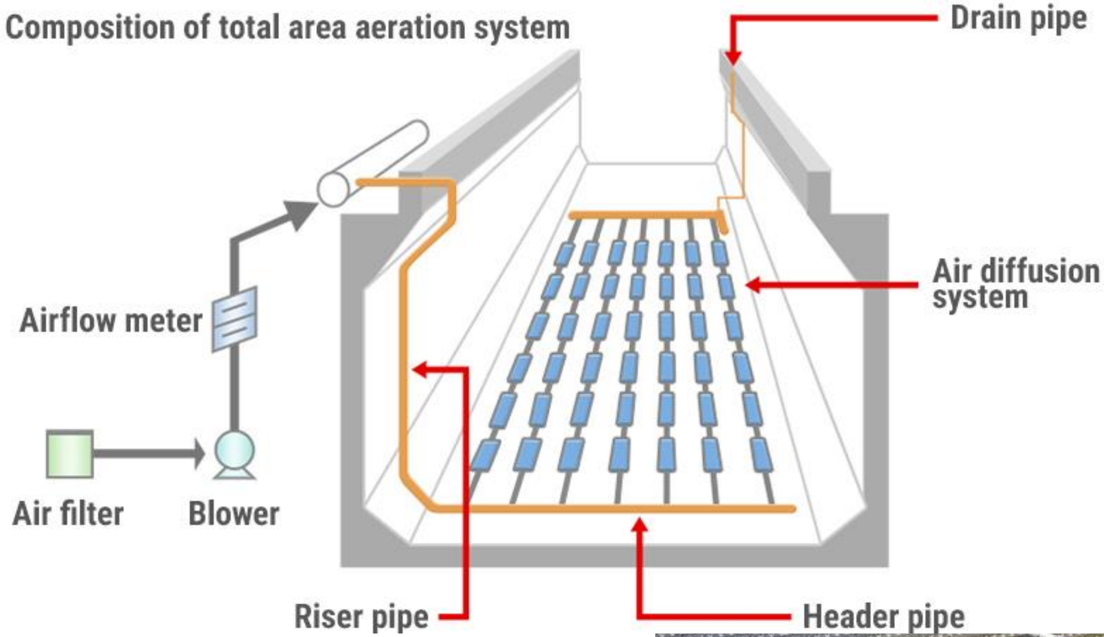
Oxygen requirements= Oxygen demand + mixing requirement

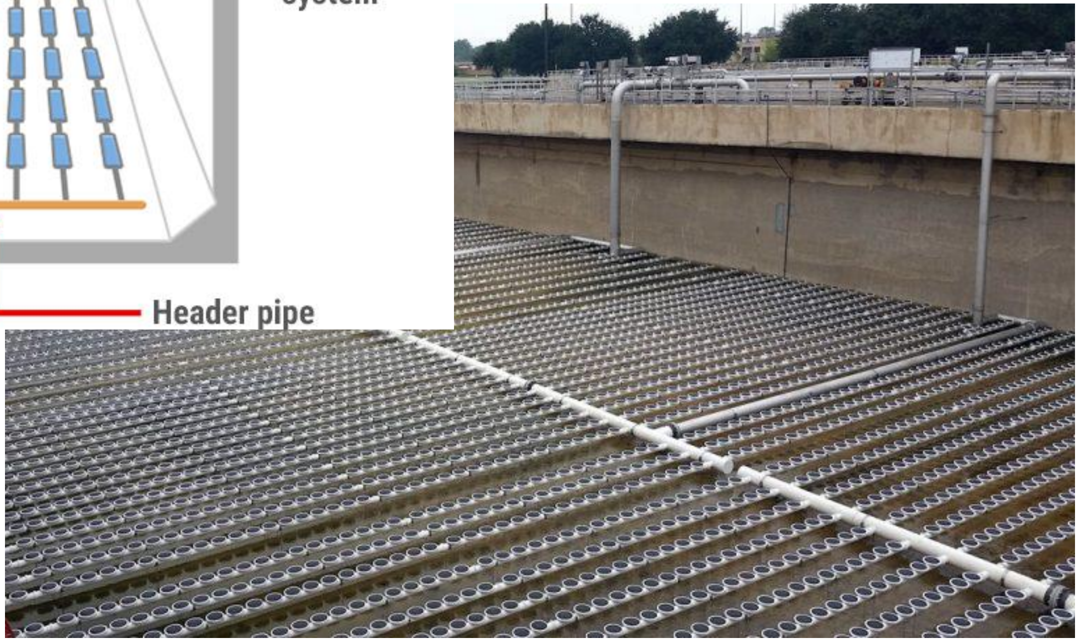
Mixing requirements

- **In diffused aeration: 20-30 m³/(min·1000 m³)**
- **In mechanical aerator: 20-40 W/1000 m³**

Oxygen transfer and mixing

Design of diffused aeration systems

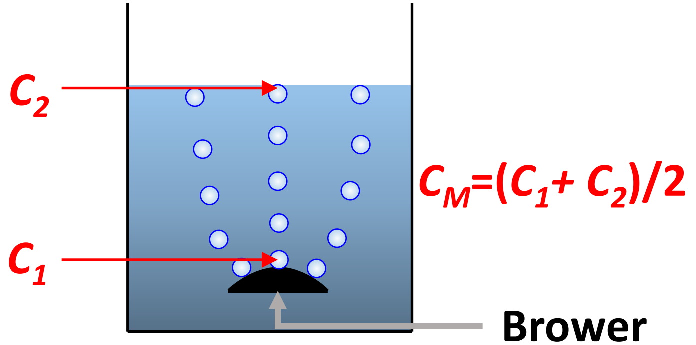




Oxygen transfer and mixing

Design of diffused aeration systems

$$ATOR = STOR\frac{\alpha\,(C_M - C_t)(\theta)^{(T-20°C)}}{9.17}$$



The DO saturation at mid-depth, mg/L

$$C_M = \beta C_s\left(\frac{P_r}{C} + \frac{O_e}{42}\right)$$

C_s = dissolved oxygen saturation concentration at the surface of the basin at a specified temperature and elevation or pressure, mg/L
P_r = absolute pressure at the depth of bubble release, kPa
O_e = percent oxygen content in the exit air flow, %
C = 203 for SI units.

Oxygen transfer and mixing

Example 7.13 Design of diffused aeration system

Use the data and answers from Example 7.12 to design a coarse-bubble diffused aeration system. Aeration basin volume = 5500 m^3; assume α and β coefficients for the diffusers are 0.40 and 0.95, respectively and use a temperature correction factor (θ) of 1.024. The dissolved oxygen (DO) saturation concentration of water is 7.54 mg/L at sea level and 30°C. A DO concentration of 2.0 mg/L is maintained in the aeration basin. The oxygen transfer efficiency is assumed to be 10% since coarse-bubble diffusers are being used. The side water depth of the aeration basin is 4.5 m with diffusers placed 0.3 m from the bottom of the tank. The barometric pressure is 101.3 kPa and the specific weight of water is 1000 kg/m^3. Assume the density of air at seal level at 20°C is ρ_{air} = 1.2 kg/m^3. Determine:

a) Volume of the air (m^3/min) to meet oxygen requirement of 6800 kg/d.

Oxygen transfer and mixing

a) Volume of the air (m^3/min) to meet oxygen requirement of 6800 kg/d.

The DO saturation concentration at mid-depth (***C_M***) must be calculated

$$C_M = \beta C_s(\frac{P_r}{C} + \frac{O_e}{42})$$

The oxygen concentration (%) in the off-gas should be calculated.
Assuming 100 moles of air to be compressed,

- 21 moles of oxygen (n_{O2})
- 79 moles of nitrogen (n_{N2})

$$n_{O_2} = 21 \text{ mole } \times (1 - OTE)$$

OTE: Oxygen Transfer Efficiency, 10%

$$n_{O_2} = 21 \text{ mole } \times (1 - 10\%) = 18.9 \text{ mole}$$

$$O_e = \frac{n_{O_2}}{n_{O_2} + n_{N_2}} = \frac{18.9}{18.9 + 79} = 0.193 = 19.3\%$$

Oxygen transfer and mixing

a) Volume of the air (m^3/min) to meet oxygen requirement of 6800 kg/d.

The DO saturation concentration at mid-depth (***C_M***) must be calculated

$$C_M = \beta C_s (\frac{P_r}{C} + \frac{O_e}{42})$$

The pressure at the point of air release (***P_r***) at the bottom of the aeration basin is calculated.

$$P_r = \rho g h + P_{bar}$$

$$P_r = (4.5 - 0.3)\mathrm{m} \times 9.8 \frac{N}{kg} \times 1000 \frac{\mathrm{kg}}{\mathrm{m}^3} + 101.3\ \mathrm{kPa}$$

$$= 41.6 \times 10^3 \frac{N}{m^2} + 101.3\mathrm{kPa} = 41.6\ kPa + 101.3\mathrm{kPa} = 142.9\ \mathrm{kPa}$$

$$C_M = \beta C_s \left(\frac{P_r}{C} + \frac{O_e}{42}\right) = 0.95 \times 7.54 \frac{mg}{L} \times \left(\frac{142.9\ \mathrm{kPa}}{203\ \mathrm{kPa}} + \frac{19.3\%}{42\%}\right)$$

$$\boxed{C_M = 8.33\ \mathrm{mg/L}}$$

Oxygen transfer and mixing

a) Volume of the air (m^3/min) to meet oxygen requirement of 6800 kg/d.

$$ATOR = STOR\frac{\alpha\,(C_M - C_t)(\theta)^{(T-20℃)}}{9.17}$$

$$6800\ \mathrm{kg/d} = STOR\frac{0.4\times\ (8.33-2)\mathrm{mg/L}\times(1.024)^{(30℃-20℃)}}{9.17}$$

$$6800\ \mathrm{kg/d} = STOR \times 0.35$$

$$\boxed{STOR = 19429\ \mathrm{kg/d}}$$

In 1kg air, oxygen is 0.23 kg.

$$\mathrm{Air\ requirement} = \frac{STOR}{0.23}\frac{1}{\rho_{air}} = \frac{19429\ \mathrm{kg/d}}{0.23}\frac{1}{1.2\ \mathrm{kg/m^3}}$$

$$\boxed{\mathrm{Air\ requirement} = 70395\ \ \mathrm{m^3/d} = 48.9\ \ \mathrm{m^3/min}}$$

$$\mathrm{Mixing\ requirement} = \frac{\mathrm{Air\ requirement}}{\mathrm{Basin\ volume}} = \frac{48.9\ \ \mathrm{m^3/min}}{5.5\times 1000\mathrm{m^3}}$$
$$= 8.9\,\mathrm{m^3/(min\cdot 1000m^3)}$$

- **In diffused aeration: 20-30 m³/(min·1000 m³)**

Oxygen transfer and mixing

Brief summary

Oxygen transfer and mixing

- Two-film theory
- Derivation of mass transfer equation
- Types of aeration systems
- Derivation of oxygen transfer equation
- Design of diffused aeration systems

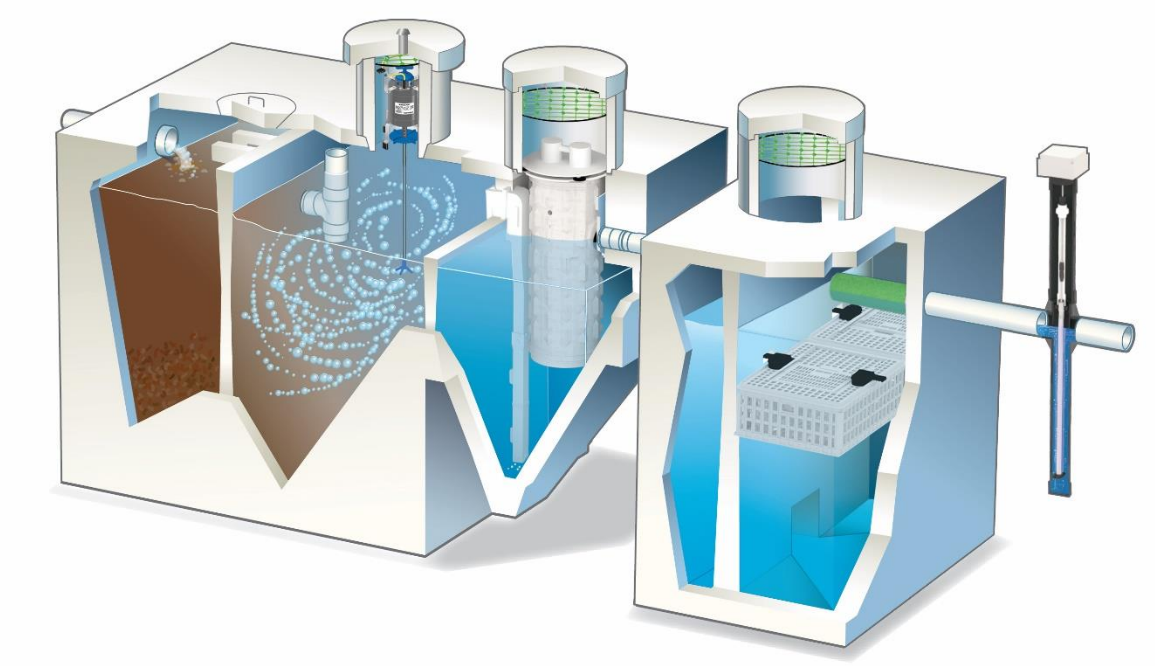
Section 2

Advanced biological treatment systems

Advanced biological treatment systems

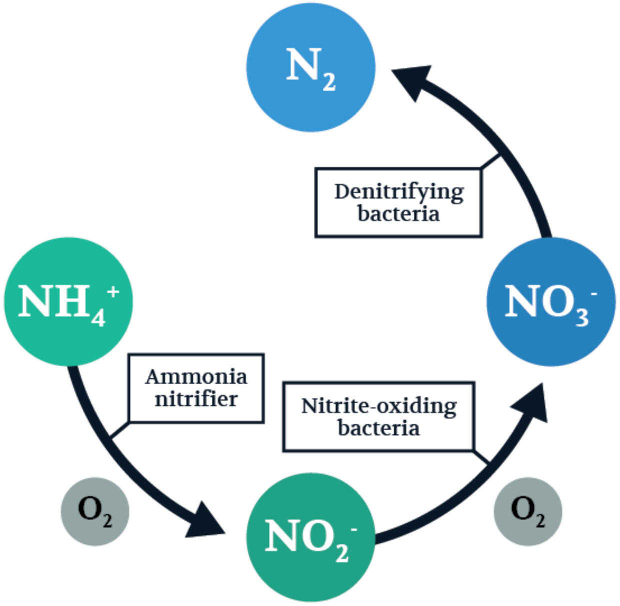
Advanced wastewater treatment (AWT) systems are designed to remove nitrogen and phosphorus, along with additional BOD and TSS, from the wastewater.

Biological nutrient removal (BNR) processes use various types of microorganisms to remove nitrogen and phosphorus beyond that which can be accomplished in conventional activated sludge systems.



Biological nitrogen removal

Biological nitrogen removal is accomplished by ***nitrification***, an aerobic process by which nitrifying autotrophic bacteria oxidize ammonia to nitrite, and ultimately to nitrate, is followed by ***denitrification***, an anoxic process wherein heterotrophic bacteria reduce nitrates produced during nitrification to the end product, nitrogen gas.



Advanced biological treatment systems

Nitrification

Nitrification is principally accomplished by two autotrophic aerobic bacterial genera, ***Nitrosomonas*** and ***Nitrobacter***. Autotrophs expend energy to fix and reduce inorganic carbon, which leads to lower yields.

$$\mathbf{NH_4^+ + 1.815O_2 + 0.1304CO_2}$$
$$\mathbf{\rightarrow 0.0261C_5H_7O_2N + 0.973NO_3^- + 0.091H_2O + 1.973H^+}$$

When 1 gram NH_4^+-N is removed, 4.15 gO_2 and 7.05 g alkalinity are consumed.

$$\left(\frac{1.815 \times 32\ \mathrm{g/mol}}{14\ \mathrm{gNH_4^+ - N/mol}}\right) = \mathbf{4.15\,\frac{gO_2}{gNH_4^+ - N}}$$

$$\left(\frac{1.973 \times 1\ \mathrm{gH^+/mol}}{14\ \mathrm{gNH_4^+ - N/mol}}\right)\left(\frac{50\ \mathrm{gCaCO_3}}{1\mathrm{gH^+}}\right) = \mathbf{7.05\,\frac{g\ akalinity\ as\ CaCO_3}{gNH_4^+ - N}}$$

Denitrification

Denitrification is accomplished by heterotrophic, denitrifying microorganisms reducing nitrate at ***anoxic environment*** to nitrogen gas that is released into the atmosphere.

$$\mathbf{6NO_3^- + 5CH_3OH \rightarrow 3N_2 + 6OH^- + 5CO_2 + 7H_2O}$$

When 1 gram NO_3^--N is removed, 1.9 gCH_3OH is consumed and 3.57 g alkalinity are produced.

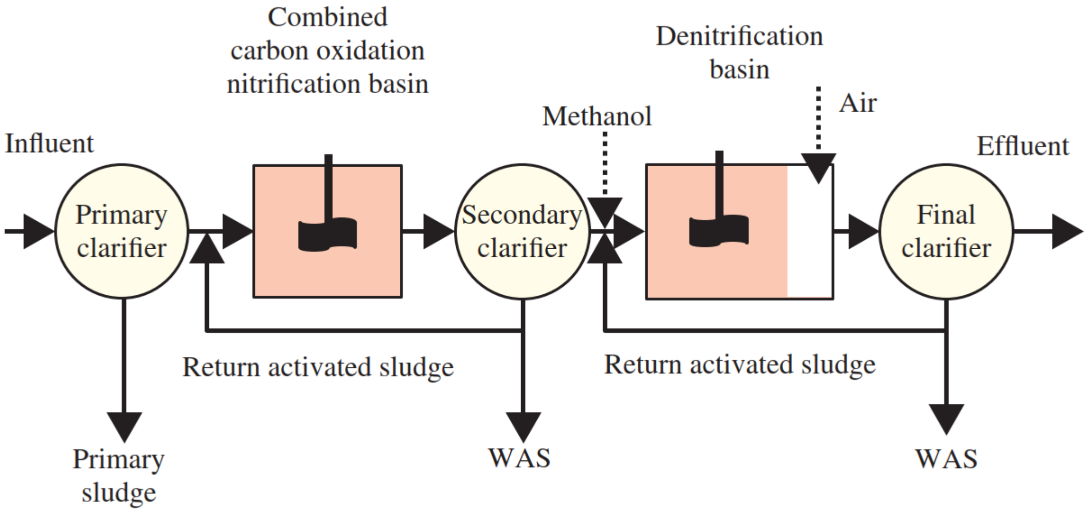
$$\frac{5 \times (32\ \mathrm{gCH_3OH})}{6 \times (14\ \mathrm{gNO_3^- - N})} = \mathbf{1.9\,\frac{g\mathit{CH_3OH}}{gNO_3^- - N}}$$

$$\frac{6 \times (17\ \mathrm{gOH^-})}{6 \times (14\ \mathrm{gNO_3^- - N})}\left(\frac{50\ \mathrm{gCaCO_3}}{17\mathrm{gOH^-}}\right) = \mathbf{3.57\,\frac{g\ akalinity\ as\ CaCO_3}{gNO_3^- - N}}$$

Advanced biological treatment systems

Biological nitrogen removal process

- **Two-stage nitrification – denitrification process**



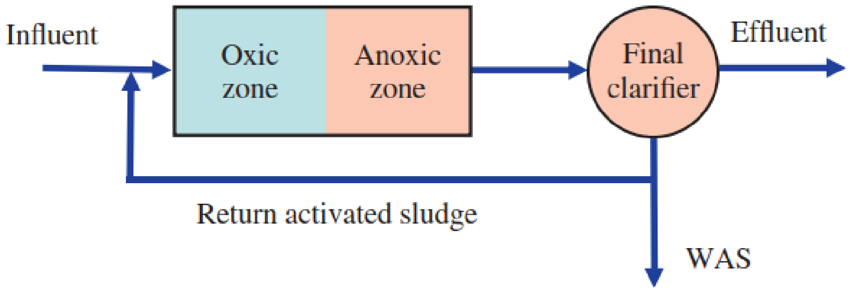
- Optimization of each stage
- Better control of the process

- large reactor volumes
- Methanol is added

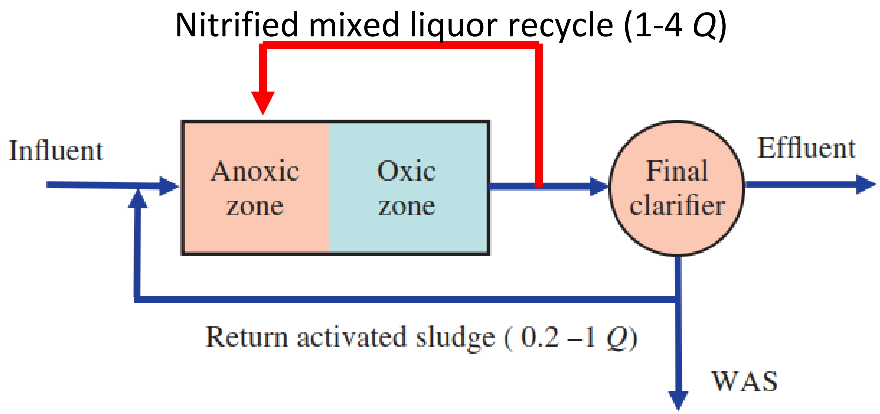
Advanced biological treatment systems

Biological nitrogen removal process

- **Post-Denitrification process**



- **Pre-Denitrification process**

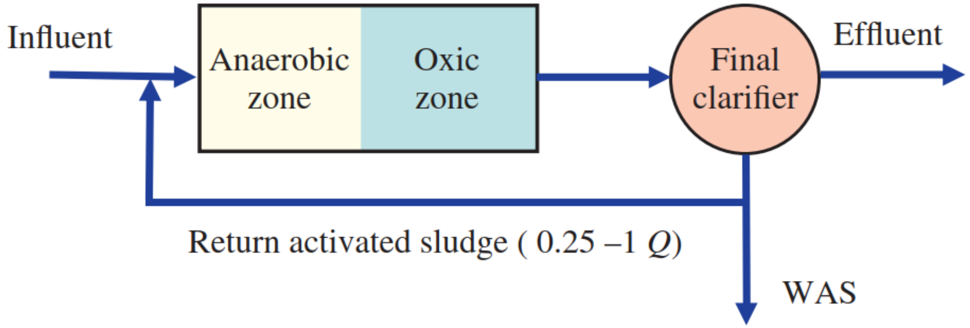


- Less O_2 required
- Less methanol required

Advanced biological treatment systems

Phosphorus removal systems

- To accomplish ***enhanced biological phosphorus removal (EBPR)***, an anaerobic zone is necessary for the fermentation of incoming organic matter into volatile fatty acids (VFAs), which are required for the growth and proliferation of phosphorus-accumulating organisms (**PAOs**) such as *Acinetobacter*.
- PAOs are capable of removing excess quantities of phosphorous.

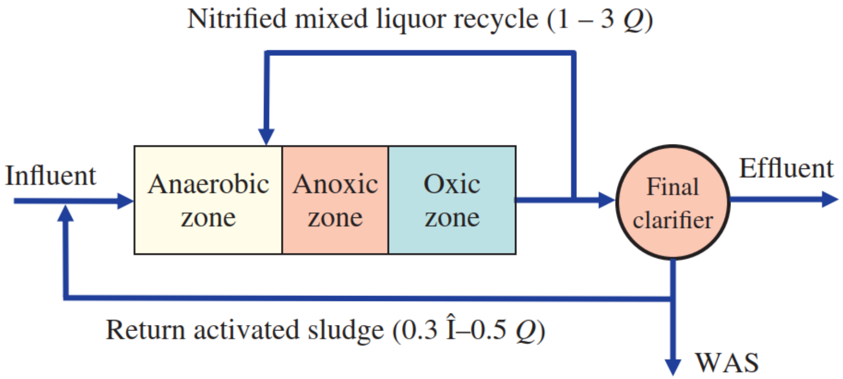


A/O biological phosphorus removal process

Advanced biological treatment systems

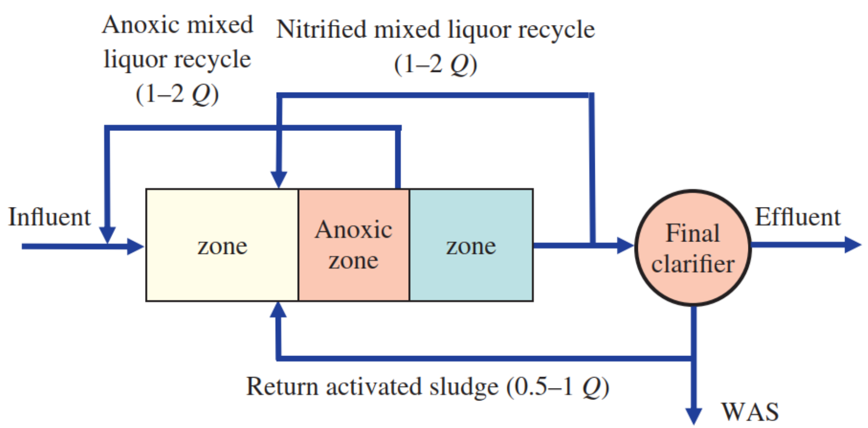
Combined biological nitrogen and phosphorus removal processes

- **A²/O process**



In the effluent
TN=6-12 mg/L
TP=0.5-4.6 mg/L

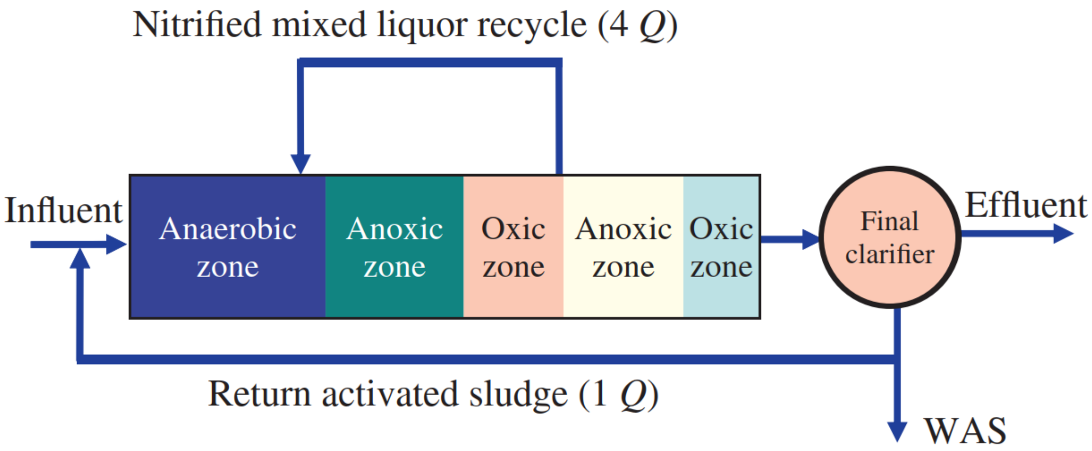
- **University of Cape Town (UCT) process**



Advanced biological treatment systems

Combined biological nitrogen and phosphorus removal processes

- **Five-Stage Bardenpho Process**



In the effluent
TN<3.0 mg/L
TP<1.0 mg/L

Advanced biological treatment systems

Brief summary

Advanced biological treatment systems

- Biological nitrogen removal
- Phosphorus removal systems
- Combined biological nitrogen and phosphorus removal processes

Summary

1. **Wastewater treatment plants** (WWTPs) consist of a series of unit operations and unit processes for treating the liquid wastewater and sludge.
2. The design engineer must know the **influent** wastewater characteristics and **effluent** requirements to select and design the unit operations and processes.
3. Major **unit operations** include: screens; grit removal; primary sedimentation.
4. Major **unit processes** include: the activated sludge process and associated modifications and SBR.
5. The capacity of a WWTP is based on both a **hydraulic flow** and process **loading**.
6. The design of primary treatment: screens; grit removal; primary sedimentation.
7. Biological wastewater treatment is the removal of organics and nutrients from wastewater by **microorganisms**.
8. The design of an activated sludge process (effluent concentration, aeration basin volume, sludge production and oxygen requirement)
9. The design of diffused aeration systems was presented.
10. Biological nitrogen removal systems use the **nitrification** process and the **denitrification** process.
11. Enhanced biological phosphorus removal (EBPR) systems are those that are configured to have alternating anaerobic-aerobic treatment.

Homework

Define the following terms:

1. **Design loadings**
2. **Detention time**
3. **Overflow rate**
4. **Activated sludge process**
5. θ
6. **WAS**
7. **Sequencing batch reactors**
8. **oxygen transfer coefficient**
9. **Biological nutrient removal**
10. **Nitrification**
11. **Denitrification**
12. **VFA**
13. ***VSS***
14. **PAOs**
15. **A^2/O process**

Problem 9, Page 439

9 A conventional activated sludge process treats 3,785 m^3/d of wastewater, containing 250 g/m^3 BOD_5, and produces an effluent containing 20 g/m^3 BOD_5. The nominal detention time in the aeration basin excluding the return activated sludge flow is six hours and the MLSS concentration is 3,000 g/m^3. Determine the following:

a. The aeration basin volume (m^3).
b. F/M ratio (d^{-1}).
c. Specific substrate utilization rate (d^{-1}).
d. Substrate removal efficiency (%).

Homework

Problem 12, Page 440

A complete-mix activated sludge process (CMAS) is to be designed to treat 22000 m^3/d of domestic wastewater having a BOD_5 of 180 mg/L. The NPDES permit requires that the effluent BOD_5 and TSS concentrations should be 20 mg/L or less on an annual average basis (effluent soluble BOD_5 should be less than 5 mg/L). The following biokinetic coefficients obtained at 20°C will be used in designing the process: Y = 0.6 mg VSS/mg BOD_5, k=4 d^{-1}, K_s=70 mg/L BOD_5, and k_d = 0.05 d^{-1}. Assume that the MLVSS concentration in the aeration basin is maintained at 2500 mg/L and the VSS : TSS ratio is 0.75. Determine the following:

a) Mean cell residence time necessary to meet the NPDES permit in 20°C.
b) Volume of the aeration basin in m^3.
c) Oxygen requirements assuming no nitrification in 20 °C.
d) The quantity of excess biomass produced in terms of TSS.

Problem 20 (a), Page 441

20 Design a coarse-bubble diffused aeration system to meet an oxygen demand of 10,000 pounds per day. The plant is located at an elevation of 1,000 feet. There are two aeration basins with a total volume equal to 1.43 MG. Assume the alpha and beta coefficients for the diffusers are 0.45 and 0.98, respectively and use a temperature correction factor (θ) of 1.024. The dissolved oxygen (DO) saturation concentration of water is 7.54 mg/L at sea level and 30°C. A DO concentration of 2.0 mg/L is maintained in the aeration basins. The oxygen transfer efficiency is assumed to be 10%, since coarse-bubble diffusers are being used. The side water depth of the aeration basin is 15 ft with diffusers placed one foot from the bottom of the tank. The atmospheric pressure at an elevation of 1,000 feet is 733 mm Hg. Assume that the specific weight of water is 62.4 lb/ft^3. Determine:

a. Volumetric flow rate of the air (scfm) to meet oxygen requirement of 10,000 lb per day.

The end