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# Περιβαλλοντική Βιοτεχνολογία- Environmental Biotechnology

Ενότητα 5: Activated Sludge

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Τμήμα Χημικών Μηχανικών

# Characteristics of the process

## Microbiology

### Types of microorganisms :

- Prokaryotes (bacteria)
- Eukaryotes (protozoa, nematodes etc.)

### Creation of flocs

- Organic polymers and electrostatic forces

**Primary consumers** of organic wastes (BOD) → heterotrophic bacteria (Gram-negative)

Consume a variety of different organic compounds.

### **Secondary consumers :**

Prokaryotes → by-products of BOD degradation and from the death and lysis of other organisms.

Eukaryotes → bacteria and bacteriophage

Sludge retention time:

4 d (S.F. 36) – 11 d (S.F. 100)



# Characteristics of the process

## Sludge retention time

### Affects to:

- soluble effluent quality
- sludge settling characteristics

**Common range** : 4 - 10 d (balance between BOD removal and economics)

### Retention time shorter than 10 d

- Ciliated protozoa with stalks can attach to the flocs and become important predators. They help to polish the effluent by drawing in with their cilia the few free bacteria in the system and consuming them.

### Retention time longer than 10 d

- Slower growing predators come into the picture, such as rotifers and nematodes which tend to eat at the floc particles themselves.
  - lead to the formation of small nonflocculating particles.
- Cause operational problems, such as sludge bulking and foaming.
- Favors the nitrifying bacteria.



# Process configurations

## System of activated sludge

- Aerobic reactor
- Sedimentation tank
- Recirculation of solids
- Removal of excess sludge

**Separation of hydraulic retention time ( $\theta$ ) from solid retention time ( $\theta_x$ )**



# Process configurations

## Plug-Flow character :

- Several completely mixed tanks (CSTR) in series
- Sequencing batch reactor (SBR)

The **contact-stabilization process** permits high efficiency treatment to occur in a significantly reduced total reactor volume.

- $\vartheta$  in contact reactor : 15 - 60 min
- For  $\vartheta_x$  design: the concentration of MLSS is much higher in the stabilization tank than in the contact tank.
- Requires substantially more operational skill and attention.

The **selector** tank used to overcome the problem with sludge bulking.

- (10 – 30 min contact time)



# Loading modifications

## Conventional loading → 85% removal of BOD<sub>5</sub> and suspended solids (SS)

- HRT = 6 h
- BOD : 0.56 kg/m<sup>3</sup>-d
- MLSS : 1,200 – 3,000 mg/l

## Modified aeration → < 65-75% removal of BOD<sub>5</sub> and SS

- HRT = 1.5 – 2 h
- MLSS : 300 – 600 mg/l

## High rate → 90% removal of BOD<sub>5</sub> and SS

- BOD : 1.6 – 3.2 kg/m<sup>3</sup>-d
- MLSS : 4,000 – 10,000 mg/l
- It requires better oxygen transfer (more air) and sludge settling can be avoided.

## Extended aeration → 85% removal of BOD<sub>5</sub> and SS

- HRT = 24 h
- BOD < 0.24 kg/m<sup>3</sup>-d
- MLSS : 3,000 – 6,000 mg/l



# Ratio F/M

$$F / M = \frac{Q^0 S^0}{V X}$$

in which

F/M = food-to-microorganism ratio, kg BOD or COD applied per day per kg of total suspended solids in the aeration tank

$Q^0$  = influent wastewater stream flow rate ( $\text{m}^3/\text{d}$ )

$S^0$  = influent wastewater concentration (BOD or COD in mg/l)

$V$  = aeration-tank volume ( $\text{m}^3$ )

$X$  = total suspended solids concentration in aeration tank (mg/l)

## Conventional loading:

$F/M = 0.25 - 0.5 \text{ kg BOD}_5/\text{kg MLSS} \cdot \text{d}$

## High rate:

$F/M = 1 - 4 \text{ kg BOD}_5/\text{kg MLSS} \cdot \text{d}$

## Extended aeration:

$F/M = 0.12 - 0.25 \text{ kg BOD}_5/\text{kg MLSS} \cdot \text{d}$



# Ratio F/M

Definition of the volumetric loading for a CSTR: 
$$\frac{\hat{q}S}{K+S} X_a = \frac{Q^0(S^0 - S)}{V}$$

It can be rearranged to give the following form: 
$$\frac{Q^0 S^0}{V X_a} = S \left( \frac{\hat{q}}{K+S} + \frac{Q^0}{V X_a} \right)$$

Of the two ratios within the brackets on the right side of the equation, the left term is far larger than the right term when  $S$  is small. Indeed, if  $S$  is small with respect to  $K$ , then the left term about equals  $\hat{q}/K$ . Thus, for the usual case in which we have high treatment efficiency and a low effluent BOD concentration (typical of activated sludge treatment), above equation can be simplified to:

$$F / M_a = \frac{\hat{q}}{K} S \Rightarrow S \approx \frac{K}{\hat{q}} \cdot F / M_a$$

**The units of F/M are almost the same as those of the maximum specific utilization rate,  $q_m$ .**

**To have a good safety factor, we must have  $(F/M_a)/q_m \ll 1$**





# Loading rates

The F/M ratio is inversely proportional to the SRT, and this is completely logical since the F/M ratio is a crude approximation of the cells' specific substrate-utilization rate,  $U$ .  $U$  is related to  $\vartheta_x$  by:

$$\vartheta_x = (YU - b)^{-1} \quad \text{where} \quad U = -\frac{r_{ut}}{X_a} = \frac{Q(S^0 - S)}{V X_a}$$

**A high volumetric loading to an aeration tank can be achieved in two ways.** The first is to have a small SRT, as is the case with modified aeration. The second is to maintain a high MLSS, as is the case with high-rate activated sludge and contact stabilization. No matter how the high volumetric loading is achieved, it is successful only when the aeration capacity is sufficient to meet the high volumetric oxygen uptake rate.



# MLSS, SVI and recycle ratio

## High value for $X$ in the aeration tank:

- Lower construction costs (lower aeration basin)
- Bigger settling tank (greater solids flux)
- Increased cost of the aeration system
- Increased recycle sludge rate from settling tank
- May lead to higher effluent suspended solids (SS) and BOD



# MLSS, SVI and recycle ratio

$$Q^i X = Q^e X^e + Q^s X^s$$

For a good-functioning settling tank and activated sludge system, the effluent suspended solids concentration  $X^e$  is very low, and over 99% of the suspended solids entering the settling tank settle and are removed from the bottom of the settling tank. Thus,  $Q^e X^e$  should be very small relative to the other terms in previous equation and can be taken as zero for this analysis. Also, the sludge wasting flow rate,  $Q^w$ , normally is only a few percent at most of the recycle flow rate  $Q^r$ . Thus, reasonable approximations are:

$$Q^i X = Q^s X^s \quad (X^e \rightarrow 0)$$

$$Q^r = Q^s \quad (Q^w \rightarrow 0)$$

and

$$X^r = X^s$$



# MLSS, SVI and recycle ratio

$$X = X_m^r \frac{R}{1+R} \quad \text{or} \quad R = \frac{X}{(X_m^r - X)}$$

where

$$R = \frac{Q^r}{Q^0} = \text{the recycle ratio}$$

$X_m^r$  can be approximated for given sludges through relatively simple tests, such as the Settled Sludge Volume Test, the Sludge Volume Index (SVI).

$$SVI \text{ (ml / g SS)} = V_{30} \cdot (1,000 \text{ mg / g}) / [(MLSS) \cdot (V_t)]$$

A first approximation to the maximum concentration of settled sludge ( $X_m^r$ ) is  $(10^6 \text{ mg} \cdot \text{ml/g} \cdot \text{l})/\text{SVI}$ . Then, a typical good SVI, 100 ml/g, corresponds to an  $X_m^r$  of 10,000 mg/l. A bulking sludge would have an SVI of 200 or more, corresponding to an  $X_m^r$  of 5,000 mg/l or less, and a highly compact and good-settling sludge would have an SVI of 50 or less, corresponding to a  $X_m^r$  of 20,000 mg/l or more.

$$X_m^r = X_m^r \frac{R}{1+R}$$

**$X_m^r$  : the maximum value of  $X^r$**   
**(10,000 – 20,000 mg/l)**  
**Bulking : 3,000 – 6,000**



# Eckenfelder & McKinney Equations

The predicted effluent BOD concentrations (S) are:

Eckenfelder: 
$$S = \frac{S^0}{1 + k_E X_E \mathcal{G}} \quad (S \ll K)$$

McKinney: 
$$S = \frac{S^0}{1 + k_M \mathcal{G}} \quad (S \ll K)$$

The rate equations behind the solutions relate to the Monod terms by :

$$k_E = \frac{\hat{q}}{K} \cdot \frac{X_a}{X_v}$$

$$k_M = \frac{\hat{q}}{K} \cdot X_a$$



# Oxygen-transfer in aeration systems

$$r_{O_2} = K_L a (c_l^* - c_l)$$

where

$R_{O_2}$  = oxygen transfer rate per unit volume of reactor ( $\text{mg l}^{-1} \text{d}^{-1}$ )

$K_L a$  = volumetric mass transfer rate coefficient ( $\text{d}^{-1}$ ),

$c_l^*$  = liquid phase oxygen concentration in equilibrium with bulk gas phase ( $\text{mg/l}$ ),

$c_l$  = liquid phase bulk oxygen concentration ( $\text{mg/l}$ ).

$$c_l^* = \frac{c_g}{H_{O_2}}$$

Henry's constant depends strongly on temperature. For clean water, the relationship is given approximately by:

$$\log H_{O_2} = 0.914 - \frac{750}{T_K}$$



# Oxygen-transfer in aeration systems

**Saturation oxygen concentration:** Determine  $c_l^*$  for aeration of clean water with air in Denver if the atmospheric pressure is 0.8 atm and the wastewater temperature is 12°C.

$$\log H_{O_2} = 0.914 - \frac{750}{273 + 12} = -1.718$$

$$H_{O_2} = 10^{-1.718} = 0.0192 \text{ atm} \cdot \text{l} \cdot \text{mg}^{-1}$$

$$c_g = 0.2095(0.8) = 0.168 \text{ atm}$$

$$c_l^* = \frac{0.168}{0.0192} = 8.7 \text{ mg / l}$$

Oxygen solubility in wastewater often deviates from that given by the combination of previous equations (previous slide) due to the effect of salts and organic material. In order to correct for this effect, the clean water solubility for oxygen is multiplied by a value  $\beta$ :

$$\beta = \frac{c_l^*(\text{wastewater})}{c_l^*(\text{clean water})}$$

Typically,  $\beta$  varies from 0.7 to 0.98, and a value of 0.95 is often assumed for municipal wastewaters.



# Oxygen-transfer in aeration systems

To express this difference, a factor  $\alpha$  is used:

$$\alpha = \frac{K_L \alpha (\text{wastewater})}{K_L \alpha (\text{clean water})}$$

The value for  $\alpha$  is reported to vary from 0.35 to 0.8 for diffused aeration and from 0.3 to 1.1 for mechanical aeration.

**$K_L \alpha$  is affected by wastewater characteristics.**

Regardless of the aeration type, the power input per unit volume is a key factor that affects  $K_L a$ . The amount of power dissipated determines the scale of the eddies and the intensity of turbulent velocity fluctuations at length scales comparable to bubble sizes.

In line with this observation, oxygen transfer is generally expressed as mass of oxygen transferred under standard conditions per unit of power input to the aerator. **Standard oxygen transfer efficiencies (SOTE)** are for standard conditions of 20°C ( $c_l^* = 9.2$  mg/l), zero oxygen in the liquid phase ( $c_l = 0$  mg/l), and with clean water ( $\alpha$  and  $\beta = 1$ ). SOTEs generally are 1.2 to 2.7 kg O<sub>2</sub> per kWh.





# Oxygen-transfer in aeration systems

For a design, the energy efficiency must be converted to field conditions. The prevailing temperature,  $\alpha$ ,  $\beta$ ,  $c_l$ , and  $c_l^*$  must be specified. Then, the SOTE (e.g., 2 kg O<sub>2</sub> per kWh) can be converted to the **field oxygen transfer efficiency (FOTE)** by:

$$FOTE = SOTE \cdot 1.035^{T-20} \cdot \alpha \cdot (\beta c_l^* - c_l) / 9.2$$

9.2 is the mg/l O<sub>2</sub> saturation concentration for 20°C and  $c_g = 0.21$  atm O<sub>2</sub>.  $1.035^{T-20}$  is a typical correction factor for the effects of temperature on mass-transfer kinetics. For example, an SOTE of 2 kg O<sub>2</sub> per kWh is reduced to an FOTE of 0.74 kg O<sub>2</sub> per kWh when  $T = 12^\circ\text{C}$ ,  $c_g = 0.8(0.21)$  atm,  $c_l^* = 8.7$  mg/l,  $c_l = 2$  mg/l,  $\alpha = 0.7$ , and  $\beta = 0.95$ . This example illustrates the normal trend: FOTE is considerably smaller than SOTE. Manufacturers of aeration equipment normally report SOTE values.

**a reasonable value to assume for preliminary calculations is 1 kg O<sub>2</sub> per kWh.**



# Oxygen-transfer in aeration systems

Aeration is also used to maintain the microbial flocs in suspension. The Joint Manual of Practice by ASCE and the WEF suggests that **mixed-liquor velocities** in an aeration tank equal at least **0.3 m/s**. This requires an air input for diffused air systems of 20 to 30 m<sup>3</sup> air per min per 1,000 m<sup>3</sup> of tank volume. For mechanical mixing with vertical mixing regime, a power input of 15 to 30 kW per 1,000 m<sup>3</sup> is required. Generally, the limiting factor in aerator design is oxygen demand, rather than mixing, when air is used as the source of oxygen. However, with extended aeration systems or when pure oxygen is used to supply the oxygen demand, the mixing requirement may govern.



# Bulking sludge

Fundamental and applied research have identified three distinct causes of **sludge bulking: low dissolved oxygen, long SRT, and input reduced sulfur**. We discuss each of the causes and how they can be overcome. The means to overcome the different types of bulking are totally distinct. Therefore, it is essential to understand which type of bulking is occurring before designing a solution strategy.

To prevent bulking, the D.O. concentration must be greater than

$$D.O. (mg / l) > (U - 0.1) / 0.22$$

in which U is in kg COD/kg MLVSS-d. The solution to overcoming low-D.O. bulking is to increase the D.O. concentration, which may require expanding the capacity of the aeration system or reducing the BOD loading.



# Bulking sludge

The **selector technique** has proven very effective for counteracting low-F/M bulking in these situations.

The selector tank is placed in front of the aeration basin. It receives the influent and the recycled sludge. It is mixed, and it may or may not be aerated. The design goals for the selector are: **(1)** it must be small enough that oxidation of the entering BOD is small; and **(2)** it must be large enough that soluble components, like organic volatile acids, are taken up rapidly by floc-forming bacteria that store internal polymers such as polyhydroxybutyrate (PHB).

$$F / M_{sel} = Q \cdot BOD_5^0 / (V_{sel} \cdot MLVSS)$$

in which  $F/M_{sel}$  has units  $g \text{ BOD}_5/g \text{ MLVSS-d}$ ,  $Q$  is the influent flow ( $m^3/d$ ),  $BOD_5^0$  is the influent  $BOD_5$  concentration ( $mg/l$ ),  $V_{sel}$  is the volume of the selector tank ( $m^3$ ), and  $MLVSS$  is the VSS concentration in the mixed liquor in the selector ( $mg/l$ ). From the literature and the authors' experience, good bulking abatement occurs for  **$F/M_{sel}$  between 30 and 40  $g \text{ BOD}_5/g \text{ MLVSS-d}$** .



# Bulking sludge

**Reduced-sulfur bulking** occurs whenever reduced sulfur compounds, mostly sulfides, enter the activated sludge unit. Sulfur-oxidizing species, such as *Thiothrix* and O21N, are filament formers that gain a competitive advantage from the chemolithotrophic electron donor. So far, the only reliable means to eliminate reduced-sulfur bulking is to eliminate all inputs of reduced sulfur. If the source cannot be eliminated, the reduced sulfur can be chemically oxidized before the wastewater enters the activated sludge process. Although a number of chemical oxidants can be used, the easiest one is hydrogen peroxide, which oxidizes sulfides according to



The reaction indicates that 4.25 g H<sub>2</sub>O<sub>2</sub> is needed to oxidize one g S. In reality, the peroxide reacts with other reduced compounds.



# References

The images where their origin is not mentioned are derived from the book:  
Environmental Biotechnology : Principles and Applications,  
Bruce E. Rittmann and Perry L. McCarty,  
McGraw-Hill Series in Water Resources and Environmental Engineering



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