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Organic matter removal

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4.1 INTRODUCTION

4.1.1 Transformations in the biological reactor

For the activated sludge system, it is necessary to characterize wastewater both physically (soluble, non-settleable (colloidal and /or suspended), settleable, organic, inorganic) and biologically (biodegradable, unbiodegradable). The physical, chemical and biological transformations of the organic and inorganic wastewater constituents that take place in the biological reactor are outlined in Figure 4.1. Some of these transformations are important for achieving the required effluent quality while others are not important for the effluent quality but are important for the system design and operation. In Figure 4.1 each of the wastewater organic and inorganic fractions have soluble and particulate fractions, the latter of which subdivides further into suspended (non-settleable) and settleable. Each of the three organic subfractions in turn has biodegradable and unbiodegradable constituents. The inorganic particulate subfraction comprises both settleable and suspended (non-settleable) constituents while the soluble inorganic

subfraction comprises both precipitable and non-precipitable and biologically utilizable and non-biologically utilizable constituents.

In the biological reactor the biodegradable organics, whether soluble, non-settleable or settleable, are transformed into ordinary heterotrophic organisms (OHOs, X_{OHO}), which become part of the organic (volatile) suspended solids (VSS) in the reactor. When these organisms die, they leave behind unbiodegradable particulate (but not soluble) organics, called the endogenous residue, comprising mainly unbiodegradable cell wall material ($X_{E,OHO}$). This endogenous residue becomes part of the VSS mass in the reactor. The unbiodegradable suspended and settleable organics from the influent become enmeshed with the OHO and endogenous residue masses. Together these three constituents ($X_{OHO} + X_{E,OHO} + X_U$) form the organic component of the suspended solids that accumulate in the biological reactor (VSS, X_{VSS}). The inorganic settleable and suspended constituents, together with the precipitable soluble inorganics, form the inorganic component of the suspended solids mass (ISS). The biologically utilizable soluble inorganics are absorbed by the biomass and become part of it or are transformed into

the gas phase, in which case they escape to the atmosphere. The non-precipitable and non-biologically utilizable soluble inorganics escape with the effluent. Because of the efficient bioflocculation capability of the organic activated sludge mass, all the solids material, whether biodegradable or nonbiodegradable, organic or inorganic, becomes settleable solids. Very little suspended or colloidal (non-settleable) solids mass is formed in the reactor, but when it does it cannot be retained in the system anyway and escapes with the effluent.

Wastewater constituents			Reaction		Sludge constituents	
Organic	Soluble	Unbiodegradable	Escapes with effluent			
		Biodegradable	Transforms to active organisms			
	Particulate	Unbiodegradable	Enmeshed with sludge mass			
		Biodegradable	Transforms to active organisms			
		Unbiodegradable	Enmeshed with sludge mass			
		Biodegradable	Transforms to active organisms			
Inorganic	Particulate	Settleable	Total settleable solids (TSS)			
		Suspended				
	Soluble	Precipitable	Transforms to settleable solids		Inorganic mass all settleable non-suspended	
		Biologically utilizable	Transfers to	Solids		Escapes as gas
				Gas		
		Non-precipitable & non biologically util.		Escapes with effluent		

Figure 4.1 Global transformation reactions of organic and inorganic wastewater constituents from the particulate and soluble forms in the solid and liquid phases to the solid phase as sludge constituents, and gas and liquid phases escaping to the atmosphere and with the effluent, respectively.

The degree of wastewater characterization required for the activated sludge system design is not only determined by the physical, chemical and biological processes taking place in the system, but also by the level of sophistication of the design procedures to be applied for design. This is determined largely by the effluent quality required in terms of organic matter (C), nitrogen (N) and

phosphorus (P). Generally, the more stringent the effluent quality requirements in terms of C, N and P, the more complex the activated sludge system has to be to achieve the required removals, and the more advanced and realistic the design procedures need to be. The more sophisticated and refined the design procedures are, the more detailed and refined the wastewater characterization needs to be (detailed wastewater fractionation is presented in Section 3.14).

For organic material removal only, with the wastewater strength measured in terms of BOD₅ and suspended solids (SS, settleable and/or non-settleable), little more than a knowledge of the organic load in terms of BOD₅ and SS is adequate. Knowledge of the kind of organics that make up the BOD₅ and SS generally is not required because various empirical relationships have been developed linking the BOD₅ and SS loads to the expected response and performance of the activated sludge system insofar as sludge production and oxygen demand are concerned. Where the organics are assessed in terms of COD, because the COD parameter includes both unbiodegradable and biodegradable organic material, an elementary characterization of the organic material is required, *i.e.* biodegradable and unbiodegradable and soluble and particulate COD concentrations need to be known. The unbiodegradable particulate COD concentration heavily affects sludge accumulation in the reactor and daily sludge production and the unbiodegradable soluble COD concentration fixes the filtered effluent COD concentration from the system. Without nitrification, N removal or P removal, no wastewater N and P characteristics are required. If nitrification is included in the system, knowledge of the components making up the N material in the influent is required (TKN and FSA). With biological nitrogen removal (denitrification), much more information is required: now not only the organic load in terms of COD (not BOD₅) needs to be specified, but also the quality and quantity of some of the organic compounds that make up the total organic (COD) load. Also, the nitrogenous (N) materials need to be characterized and quantified in the same way. With biological P removal, still further specific information characterizing the organic material is required and

additionally characterization of the phosphorous (P) materials is required. The quality and quantity of C, N and P compounds entering the nitrogen (N) and nutrient (N and P) removal activated sludge reactor are affected by some unit operations upstream of the reactors, in particular primary sedimentation. It is thus important that the effect of primary sedimentation on the wastewater C, N and P constituents is also determined, to enable the settled sewage characteristics to be estimated.

4.1.2 Steady-state and dynamic-simulation models

For mathematical modelling of wastewater treatment systems, generally two levels of mathematical models have been developed; steady state and dynamic simulation. Steady-state models have constant flows and loads and are relatively very simple. This simplicity makes these models very useful for design or evaluation of existing wastewater treatment plants (WWTPs). In these models complete descriptions of system parameters are not required, but rather the models are oriented to determining the important system design parameters from performance criteria (Ekama, 2009). Dynamic models are much more complex than steady-state ones and have varying flows and loads with the result that time is included as a parameter. Dynamic-simulation models are therefore useful in predicting time-dependent system responses of an existing or proposed system. However, their complexity demands that many more kinetic and stoichiometric constants need to be supplied and all the system design parameters have to be specified. Steady-state models are very useful for calculating the initial conditions required to start dynamic-simulation models such as reactor volumes, recycle and waste flows and values for the various concentrations in the reactor(s) and cross-checking simulation model outputs (Ekama, 2009).

4.2 ACTIVATED SLUDGE SYSTEM CONSTRAINTS

Basically all aerobic biological treatment systems operate on the same principles, *i.e.* trickling filters,

aerated lagoons, contact-stabilization, extended aeration, etc. They differ only in the conditions under which the biological reactions are constrained to operate, called system constraints. The activated sludge system comprises the flow regime in the reactor, the sizes and shape, number and configuration of the reactors, recycle flows, influent flow and other features incorporated either deliberately, or present inadvertently or unavoidably. Whereas the response of the organisms is in accordance with their nature *i.e.* biological process behaviour, that of the system is governed by a combination of the organism behaviour and the physical features which define the system, *i.e.* the environmental conditions or system constraints under which the biological processes are constrained to operate.

4.2.1 Mixing regimes

In the activated sludge system, the mixing regime in the reactor and the sludge return are part of the system constraints and therefore influence the response of the system, and hence consideration must be given to reactor mixing regimes. There are two extremes of mixing; completely mixed and plug flow (Figure 4.2).

In the completely mixed regime the influent is instantaneously and thoroughly mixed (theoretically) with the reactor contents. Hence the effluent flow from the reactor has the same compound concentrations as the reactor contents. The reactor effluent flow passes to a settling tank; the overflow from the tank is the treated waste stream, the underflow is concentrated sludge and is recycled back to the reactor. In the completely mixed system the rate of return of the underflow has no effect on the biological reactor except if an undue sludge build-up occurs in the settling tank. The shape of the reactor is approximately square or circular in plan, and mixing is usually by mechanical aerators or diffused air bubble aeration. Examples are extended aeration plants, aerated lagoons, Pasveer ditches and single reactor completely mixed activated sludge plants.

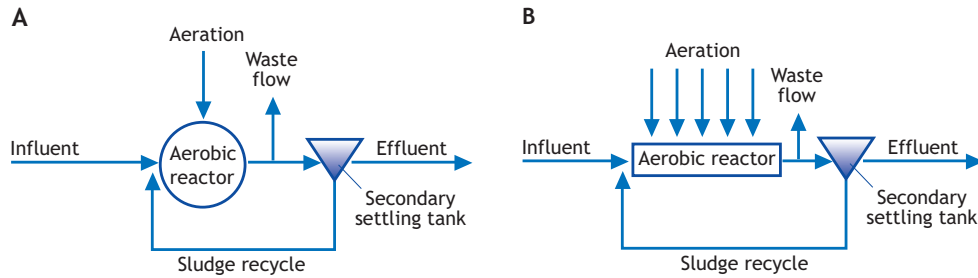


Figure 4.2 Activated sludge systems with (A) a single reactor completely mixed reactor mixing regime, and (B) a plug flow/intermediate reactor mixing regime.

In a plug-flow regime, the reactor is usually a long channel-type basin. The influent is introduced at one end of the channel, flows along the channel axis and is mixed by air spargers set along one side of the channel or horizontal shaft surface aerators. Theoretically each volume element of liquid along the axis is assumed to remain unmixed with the elements leading and following. Discharge to the settling tank takes place at the end of the channel. To inoculate the influent waste flow with organisms, the underflow from the settling tank is returned to the influent end of the channel. This creates an intermediate flow regime deviating from true plug-flow conditions depending on the magnitude of the recycled underflow. Conventional activated sludge plants are of the intermediate flow-regime type with sludge return recycle ratios varying from 0.25 to 3 times the average influent flow rate. If the recycle ratio is very high, the mixing regime approaches that of completely mixed.

Intermediate flow regimes are also achieved by having two or more completely mixed reactors in series, or by step-aeration. In the latter, the influent is fed at a series of points along the axis of the plug-flow-type reactor. Both configurations require, for inoculation purposes, recycling of the settled sludge from the settling tank(s) to the start of the channel reactor.

The mean kinetic response of an activated sludge system, *i.e.* sludge mass, daily sludge production, daily oxygen demand and effluent organics

concentration is adequately, indeed accurately, given by assuming the system is completely mixed and the influent flow and load are constant. This allows the reactor volume, the mass of sludge wasted daily, and the average daily oxygen utilization rate to be determined by relatively simple formulations. Peak oxygen utilization rates which arise under cyclic flow and load conditions can be estimated subsequently quite accurately by applying a factor to the average oxygen utilization rate. These factors have been developed from simulation studies with the simulation models on aerobic and anoxic-aerobic systems operated under cyclic and under constant flow and load conditions (Muvvoti *et al.*, 2002).

4.2.2 Sludge retention time (SRT)

In the schematic diagrams for the activated sludge system (Figure 4.2), the waste (or surplus) sludge is abstracted directly from the biological reactor. The common practice is that the waste sludge is abstracted from the secondary settling tank underflow. Sludge abstraction directly from the reactor leads to a method of control of the sludge age (sludge retention time or solids retention time: SRT), called the *hydraulic control of sludge age*, which has significant advantages for system control compared to abstracting wastage via the underflow.

The sludge age, SRT in days, is defined by:

$$\text{SRT} = \frac{\text{Mass of sludge in reactor}}{\text{Mass of sludge wasted per day}} \quad (\text{d})$$

By abstracting the sludge directly from the reactor, the sludge concentrations in the waste flow and biological reactor are the same. If sludge age of, say, 10 days is required, one tenth of the volume of the reactor is wasted every day. This can be achieved by a constant waste flow rate, Q_w (l/d), where Q_w is the volume of sludge to be wasted daily. Hence,

$$\text{SRT} = \frac{X_{\text{TSS}} \cdot V_R}{X_{\text{TSS}} \cdot Q_w} = \frac{V_R}{Q_w} \quad (\text{d}) \quad (4.1)$$

where:

V_R volume of the biological reactor (l)
 Q_w waste flow rate from reactor (l/d)

Equation 4.1 assumes that the loss of solids with the effluent is negligible and that the mass of sludge in the secondary settling tanks is also negligible relative to that in the biological reactor. This assumption is reasonable when the system is operated at relatively high underflow recycle ratios (~1:1) and the sludge age is longer than approximately 3 days (see Section 4.10).

4.2.3 Nominal hydraulic retention time (HRT_n)

In activated sludge theory, the volume of the reactor per unit of volume of influent flow is known as the nominal hydraulic retention time *i.e.*

$$\text{HRT}_n = \frac{V_R}{Q_i} \quad (\text{d}) \quad (4.2)$$

where:

HRT_n average nominal hydraulic retention time (d)
 Q_i daily average influent flow rate (l/d)

When the sludge return flow from the secondary settling tank (Q_s) and any other mixed liquor recycle flow entering the reactor (Q_a) are included, the

retention time is called the actual hydraulic retention time (HRT_a), *viz.*

$$\text{HRT}_a = \frac{V_R}{Q_i + Q_s + Q_a} = \frac{\text{HRT}_n}{1 + s + a} \quad (\text{d}) \quad (4.3)$$

where:

HRT_a actual hydraulic retention time (d)
 s sludge underflow recycle ratio (Q_s/Q_i)
 a mixed liquor recycle ratio (Q_a/Q_i)

4.2.4 Connection between sludge age and hydraulic retention time

From the above definitions, it can be seen that there are two parameters that relate to time in the system: (i) the sludge age, which gives the length of time the particulate material remains in the reactor, and (ii) the nominal hydraulic retention time, which gives the length of time the liquid and dissolved material remain in the reactor. In activated sludge systems which do not have solid liquid separation with membranes or secondary settling tanks (SSTs), such as aerated lagoons, the sludge age and nominal hydraulic retention time are equal, *i.e.* the liquid/dissolved material and the solids/particulate material remain in the reactor for the same length of time. When solid liquid separation is included, then the liquid and solid retention times are separated and $\text{SRT} > \text{HRT}_n$. However, long sludge ages lead to large sludge masses in the reactor, which in turn lead to large reactor volumes (V_R). Therefore, even with solid-liquid separation, as SRT gets longer, so also does HRT_n . This link between SRT and HRT_n is neither proportional nor linear and depends on (i) the wastewater organic (COD or BOD_5) concentration and (ii) the suspended solids concentration (TSS) in the reactor. For biological nutrient removal activated sludge systems, the sludge age is approximately 10 to 25 days and the nominal hydraulic retention time approximately 10 to 24 h.

4.3 SOME MODEL SIMPLIFICATIONS

4.3.1 Complete utilization of biodegradable organics

The distinction between biodegradable and unbiodegradable is governed by the biomass in the system and the length of time this biomass has to degrade the organics. It has been observed that the difference in the soluble effluent COD concentration from a short (2-3 h) and a very long (18-24 h) hydraulic retention time system is very small, only 10 to 20 mgCOD/l. This indicates that slowly biodegradable soluble organics seem to be very low in concentration in normal municipal wastewater. Therefore, it is reasonable to accept that the soluble organics in municipal wastewater comprise two groups: the biodegradable, which are almost all readily biodegradable and the unbiodegradable. This means that even at very short hydraulic retention times of a few hours, utilization of biodegradable organics is complete leaving only the soluble unbiodegradable organics in the effluent.

The influent particulate biodegradable organics, both settleable and suspended (X_s), are mostly slowly biodegradable. These slowly biodegradable particulate organics (SBCOD), whether settleable or non-settleable, become enmeshed within the activated sludge flocs and become part of the suspended VSS sludge mass in the reactor. As part of the sludge mass, these organics settle out with the sludge mass in the secondary setting tank and are returned to the biological reactor. Undegraded particulate organics therefore do not escape with the effluent but remain part of the sludge VSS mass in the system; the only exit route for the undegraded particulate biodegradable organics is via the waste flow (Q_w) with the waste sludge. The time available for the breakdown of the particulate slowly biodegradable organics by the OHOs is therefore related to the solids retention time or sludge age of the system. Although the biological breakdown of the particulate biodegradable organics is much slower than that of the soluble readily biodegradable organics, this is of little

consequence because the solids retention time in the system (SRT) is much longer than the liquid retention time (HRT_n). Once the sludge age is longer than approximately 3 days at 20 °C (4 at 14 °C), the slowly biodegradable organics are virtually completely utilized.

Experimental work has confirmed the above. Short sludge age, and by linked association a short hydraulic retention time, and long sludge age, and by linked association long hydraulic retention time systems yield closely similar influent unbiodegradable soluble and particulate COD fractions ($f_{SU,CODi}$ and $f_{XU,CODi}$). Hence, once the sludge age is longer than approximately 3 to 4 days, the residual biodegradable organic concentration, both soluble (S_s) and particulate (X_s), not broken down that can be accepted is very small. From this an important assumption and simplification can be made for the steady state and simulation models, *i.e.* slowly biodegradable soluble organics and very slowly biodegradable particulate organics can be assumed to be negligibly low in concentration in normal municipal wastewater. However, it must be remembered that, although reasonable, this assumption that all the biodegradable organics are degraded may not be valid for all wastewaters and depends on the type of industries in the catchment of the wastewater treatment plant. When characterizing such wastewaters, any residual biodegradable soluble and particulate organics not degraded in the system are implicitly included with the unbiodegradable soluble and particulate organic fractions, respectively, because this is the way that activated sludge models are structured. For the steady-state model, because all the biodegradable organics are utilized, an additional simplification can be made, *i.e.* it is not necessary to make a distinction between soluble and particulate biodegradable organics; all are transformed into OHO VSS mass. The steady-state activated sludge model equations below are based on this simplification.

4.4 STEADY-STATE SYSTEM EQUATIONS

Once it is recognized that all the organics in the influent, except the soluble unbiodegradable COD, are

either utilized by the OHOs to form new OHO mass via growth (X_{OHO}), or remain in the system and accumulate as unbiodegradable (inert) sludge mass ($X_{\text{E,OHO}}$ and X_{U}), it follows that the mass of sludge produced and the carbonaceous oxygen demand in the system are stoichiometric functions of the daily COD mass load; the greater the daily COD mass load, the greater the sludge production and carbonaceous oxygen demand.

The equations below give the masses of sludge generated in the reactor and wasted per day, the average daily oxygen demand and the effluent COD concentration comprising the unbiodegradable soluble organics for organic material removal as a function of the total organic (COD) load per day (FCOD_i), the wastewater characteristics, *i.e.* the unbiodegradable soluble and particulate COD fractions ($f_{\text{SU,COD}i}$ and $f_{\text{XU,COD}i}$) and the sludge age (SRT). The kinetic and stoichiometric constants in the equations, *i.e.* the specific yield coefficient (Y_{OHOv}), the specific endogenous mass loss rate (b_{OHO}), the unbiodegradable fraction of the OHOs ($f_{\text{XE,OHO}}$), and the COD/VSS ratio of the sludge ($f_{\text{cv,OHO}}$), as well as their temperature dependencies are given in Table 4.1.

4.4.1 For the influent

The mass flows or input fluxes of total organics (FCOD_i , mgCOD/d), biodegradable organics

($\text{FCOD}_{b,i}$, mgCOD/d), unbiodegradable particulate organics ($\text{FX}_{\text{Uv},i}$, mgVSS/d) and inorganic or fixed suspended solids (FSS, $\text{FX}_{\text{FSS},i}$, mgISS/d) are:

$$\text{FCOD}_i = Q_i \text{COD}_i \quad (\text{mgCOD/d}) \quad (4.4)$$

$$\text{FCOD}_{b,i} = Q_i \text{COD}_{b,i} = Q_i (S_{s,i} + X_{s,i}) \quad (\text{mgCOD/d}) \quad (4.5a)$$

$$\text{FCOD}_{b,i} = Q_i \text{COD}_i (1 - f_{\text{SU,COD}i} - f_{\text{XU,COD}i}) \quad (\text{mgCOD/d}) \quad (4.5b)$$

$$\text{FCOD}_{b,i} = \text{FCOD}_i (1 - f_{\text{SU,COD}i} - f_{\text{XU,COD}i}) \quad (\text{mgCOD/d}) \quad (4.5c)$$

$$\text{FX}_{\text{Uv},i} = Q_i X_{\text{Uv},i} \quad (\text{mgVSS/d}) \quad (4.6a)$$

$$\text{FX}_{\text{Uv},i} = Q_i f_{\text{XU,COD}i} \text{COD}_i / f_{\text{cv,UPO}} \quad (\text{mgVSS/d}) \quad (4.6b)$$

$$\text{FX}_{\text{Uv},i} = \text{FCOD}_i f_{\text{XU,COD}i} / f_{\text{cv,UPO}} \quad (\text{mgVSS/d}) \quad (4.6c)$$

$$\text{FX}_{\text{FSS},i} = Q_i X_{\text{FSS},i} \quad (\text{mgISS/d}) \quad (4.7)$$

Table 4.1 Stoichiometric and kinetic constants and their temperature dependency for the OHOs in the steady-state carbonaceous degradation activated sludge model (Marais and Ekama, 1976).

Constant	Symbol	Temperature dependency	θ	Standard value 20 °C
Yield coefficient (mgCOD/mgCOD)	Y_{OHO}	Remains constant	1	0.67
Yield coefficient (mgVSS/mgCOD)	Y_{OHOv}	Remains constant	1	0.45
Endogenous respiration rate (1/d)	b_{OHO}	$b_{\text{OHO},T} = b_{\text{OHO},20} \theta_{b,\text{OHO}}^{(T-20)}$	1.029	0.24
Endogenous residue fraction (-)	$f_{\text{XE,OHO}}$	Remains constant	1	0.2
ISS content of OHOs	$f_{\text{FSS,OHO}}$	Remains constant	1	0.15
¹ COD/VSS ratio of OHO (mgCOD/mgVSS)	$f_{\text{cv,OHO}}$	Remains constant	1	1.48
¹ COD/VSS ratio of UPO (mgCOD/mgVSS)	$f_{\text{cv,UPO}}$	Remains constant	1	1.48

¹Different COD/VSS (f_{cv}), N/VSS (f_{n}) and P/VSS (f_{p}) mass ratios can be assigned to the OHO biomass (X_{OHO}), OHO endogenous residue ($X_{\text{E,OHO}}$) and influent UPO (X_{Uv}). However, in this book these three VSS components are assigned the same f_{cv} , f_{n} and f_{p} mass ratios.

4.4.2 For the system

4.4.2.1 Reactor VSS mass

The masses of OHO VSS (MX_{OHOv} , mgVSS), endogenous residue VSS ($MX_{E,OHOv}$, mgVSS), unbiodegradable organics VSS (MX_{Iv} , mgVSS), volatile suspended solids VSS (MX_{VSS} , mgVSS) in the system are given by:

$$MX_{OHOv} = X_{OHOv} V_R \quad (\text{mgVSS}) \quad (4.8a)$$

$$MX_{E,OHOv} = X_{E,OHOv} V_R \quad (\text{mgVSS}) \quad (4.8b)$$

$$MX_{Uv} = X_{Uv} V_R \quad (\text{mgVSS}) \quad (4.8c)$$

$$MX_{VSS} = X_{VSS} V_R \quad (\text{mgVSS}) \quad (4.8d)$$

$$\begin{aligned} MX_{OHOv} &= FCOD_{b,i} \frac{Y_{OHOv} SRT}{(1 + b_{OHO,T} SRT)} \\ &= FCOD_i (1 - f_{SU,CODi} - f_{XU,CODi}) \frac{Y_{OHOv} SRT}{(1 + b_{OHO,T} SRT)} \\ &\quad (\text{mgVSS}) \quad (4.9) \end{aligned}$$

$$\begin{aligned} MX_{E,OHOv} &= f_{XE,OHO} b_{OHO,T} MX_{OHOv} SRT \\ &= FCOD_{b,i} \frac{Y_{OHOv} SRT}{(1 + b_{OHO,T} SRT)} f_{XE,OHO} b_{OHO,T} SRT \\ &= FCOD_i (1 - f_{SU,CODi} - f_{XU,CODi}) \cdot \\ &\quad \cdot \frac{Y_{OHOv} SRT}{(1 + b_{OHO,T} SRT)} f_{XE,OHO} b_{OHO,T} SRT \quad (\text{mgVSS}) \quad (4.10) \end{aligned}$$

$$\begin{aligned} MX_{Uv} &= \frac{FX_{U,i}}{f_{cv,UPO}} SRT = FX_{Uv,i} SRT \\ &= FCOD_i \frac{f_{XU,CODi}}{f_{cv,UPO}} SRT \quad (\text{mgVSS}) \quad (4.11) \end{aligned}$$

$$\begin{aligned} MX_{VSS} &= MX_{OHOv} + MX_{E,OHOv} + MX_{Uv} \\ &= FCOD_i \frac{Y_{OHOv} SRT}{(1 + b_{OHO,T} SRT)} (1 + f_{XE,OHO} b_{OHO,T} SRT) \\ &\quad + FX_{Uv,i} SRT \end{aligned}$$

$$= FCOD_i \left[\frac{(1 - f_{SU,CODi} - f_{XU,CODi}) Y_{OHOv} SRT}{(1 + b_{OHO,T} SRT)} \cdot (1 + f_{XE,OHO} b_{OHO,T} SRT) + \frac{f_{XU,CODi}}{f_{cv,UPO}} SRT \right] \quad (\text{mgVSS}) \quad (4.12)$$

4.4.2.2 Reactor ISS mass

The inorganic suspended solids (ISS) concentration from the influent accumulates in the reactor in an identical way to the unbiodegradable particulate organics (Eq. 4.12), *i.e.* the mass of influent ISS in the reactor is equal to the daily mass flow of ISS into the reactor $FX_{FSS,i}$ times the sludge age (SRT), *viz.*

$$MX_{FSS} = FX_{FSS,i} SRT \quad (\text{mgISS}) \quad (4.13a)$$

and:

$$X_{FSS,i} \quad \text{influent ISS concentration (mg ISS/l)}$$

The influent ISS is only part of the ISS that is measured in the reactor. The OHOs (and PAOs if present) also contribute to the ISS concentration. For fully aerobic and nitrification-denitrification (ND) systems, where only OHOs comprise the active biomass, the OHOs contribute approximately 10% of their OHOCOD mass (15% of their VSS mass) to the ISS (Ekama and Wentzel, 2004). It appears that this ISS mass is intracellular dissolved solids, which, when a sludge sample is dried in the TSS procedure, precipitate as ISS.

Therefore theoretically, this ISS contribution of the OHOs (and PAOs if present) to the TSS strictly should be ignored even though it manifests in the TSS test, because being intracellular dissolved solids, it does not add to the actual ISS flux in the secondary settling tank. However, because this ISS mass has always been implicitly included in the TSS test result in the past, it is retained because SST design procedures have been based on the measured TSS result. In other words, the models are adjusted to accommodate the TSS test

‘error’. Including the OHO ISS mass yields for fully aerobic and ND systems,

$$MX_{FSS} = FX_{FSS,i} SRT + f_{FSS,OHO} MX_{OHOv} \quad (4.14a)$$

$$MX_{FSS} = FX_{FSS,i} SRT + f_{FSS,OHO} f_{av,OHO} MX_{VSS} \quad (\text{mgISS/d}) \quad (4.14b)$$

where:

$f_{av,OHO}$ fraction of the VSS mass that is active OHOs, (see Section 4.4.7)
 $f_{FSS,OHO}$ inorganic content of the OHO VSS (0.15 mgISS/mgOHOVSS)

For enhanced biological phosphorus removal (EBPR) systems, the FSS in the phosphorus-accumulating organisms (PAO) also need to be included. For aerobic P uptake EBPR, $f_{FSS,PAO}$ is 1.30 mgFSS/mg PAOVSS *i.e.* 7 times higher than for OHOs when the PAO contain their maximum polyphosphate (PP) content of 0.355 mgPP-P/mgPAOVSS (Wentzel *et al.*, 1990; Ekama and Wentzel, 2004). Therefore, for EBPR systems, the VSS/TSS ratio is significantly lower than for fully aerobic and ND systems.

4.4.2.3 Reactor TSS mass

The total suspended solids (TSS) mass (MX_{TSS} , mgTSS) in the reactor is the sum of the volatile (VSS) and inorganic (ISS) suspended solids masses, *viz.*

$$MX_{TSS} = MX_{VSS} + MX_{FSS} \quad (\text{mgTSS}) \quad (4.15)$$

The VSS/TSS ratio of the sludge (f_{VT}) is

$$f_{VT} = \frac{MX_{VSS}}{MX_{TSS}} \quad (\text{mgVSS/mgTSS}) \quad (4.16)$$

If the influent FSS concentration is not known, then the reactor TSS mass (MX_{TSS}) can be calculated from an estimated VSS/TSS ratio (f_{VT}) of the sludge, *i.e.*

$$MX_{TSS} = MX_{VSS} / f_{VT} \quad (\text{mgTSS}) \quad (4.17)$$

where:

f_{VT} VSS/TSS ratio of the activated sludge

For the same influent FSS concentration ($X_{FSS,i}$), the VSS/TSS ratio of the activated sludge (f_{VT}) does not vary much with SRT even over a wide range from 3 to 30 d (Ekama and Wentzel, 2004).

4.4.2.4 Carbonaceous oxygen demand

The mass of oxygen utilized per day (FO_c , mgO₂/d),

$$\begin{aligned} FO_c &= FCOD_{b,i} \left[\frac{(1 - f_{cv,OHO} Y_{OHOv}) + (1 - f_{XE,OHO}) b_{OHO,T}}{1 + b_{OHO,T} SRT} \right] = \\ &= FCOD_i (1 - f_{SU,CODi} - f_{XU,CODi}) \left[\frac{(1 - f_{cv,OHO} Y_{OHOv}) + (1 - f_{XE,OHO}) b_{OHO,T}}{1 + b_{OHO,T} SRT} \right] \quad (\text{mgO}_2/\text{d}) \quad (4.18) \end{aligned}$$

$$FO_c = V_R O_c \quad (\text{mgO}_2/\text{l.d}) \quad (4.19)$$

where:

O_c carbonaceous oxygen utilization rate (mgO₂/l.d)

V_R volume for the reactor for a fully aerobic AS system (m³)

From Eq. 4.18, it can be seen that the mass of oxygen utilized by the OHOs per day (FO_c) is the sum of two terms. The first $(1 - f_{cv,OHO} Y_{OHOv})$ is the oxygen demand for growth of OHOs. It represents the electrons (COD) that are used in the growth process to generate energy by the OHOs to transform the utilized organics to new biomass (catabolism). The balance of the utilized electrons (COD, $f_{cv,OHO} Y_{OHOv}$) is conserved as new biomass (anabolism). It can be seen that this oxygen demand is proportional to the influent biodegradable organics and does not change with

sludge age. This is because all the influent biodegradable organics are utilized and transformed to OHO biomass. The second term is the oxygen demand for endogenous respiration, which increases as sludge age increases. The increase in carbonaceous oxygen demand (FO_c) with sludge age is therefore due to the increasing oxygen demand from endogenous respiration with sludge age. This increases because the longer the OHO VSS mass remains in the reactor, the more of this mass that is degraded via endogenous respiration, and the more of its electrons, carbon and energy that are passed to oxygen, changed to CO_2 , and lost as heat, respectively. Therefore, growth is the biological process whereby influent biodegradable organics are transformed to OHO VSS mass (anabolism) with an associated electron transfer to oxygen and an energy loss as heat (catabolism), and endogenous respiration is a process whereby the organism biodegradable organics are degraded via catabolism to CO_2 with a further oxygen demand and energy loss as heat. The electrons transferred to oxygen result in a much lower accumulation of VSS mass in the reactor compared with unbiodegradable particulate organics. All the electrons of these organics are conserved as VSS in the reactor and none are passed to oxygen. Hence the yield of unbiodegradable organics therefore is in effect 1 gCOD/gCOD or $1/f_{cv,UPO} \text{ gVSS/gCOD}$.

4.4.3 Reactor volume and retention time

Knowing the mass of total settleable solids (MX_{TSS}) in the reactor, the volume of the reactor is determined from the value specified for the MLSS concentration, X_{TSS} (Section 4.6) *i.e.*

$$V_R = MX_{TSS} / X_{TSS} \quad (l, m^3 \text{ or } Ml) \quad (4.20)$$

Knowing the volume of the reactor V_R , the nominal hydraulic retention time, HRT_n is found from the design average dry weather flow rate $Q_{i,ADWF}$ from Eq. 4.2.

4.4.4 Irrelevance of HRT

The above design equations lead to the important conclusion that hydraulic retention time (HRT_n) is irrelevant for the design of the activated sludge system. The mass of volatile suspended solids (VSS) in the reactor is a function mainly of the daily COD mass load on it and the sludge age. Similarly, the mass of TSS in the reactor is a function mainly of the daily mass loads of COD and ISS on the reactor and the sludge age. Consequently, insofar as the mass of sludge in the reactor is concerned, it is immaterial whether the mass of COD (and ISS) load per day arises from a low daily flow with a high COD (and ISS) concentration, or a high daily flow with a low COD (and ISS) concentration. Provided $FCOD_i$ (and $FX_{FSS,i}$) is the same in both cases, the mass of VSS and TSS will be virtually identical. However, the hydraulic retention times will differ, being long in the first and short in the second case, respectively. The hydraulic retention time, therefore, is incidental to the COD (and ISS) mass load, the VSS (and TSS) mass and the daily flow: it serves no basic design function for the activated sludge system. Design criteria for the activated sludge reactor volume based on hydraulic retention time should therefore be used with extreme caution because they implicitly incorporate specific wastewater strength and characteristic values typical for the regions for which they were developed.

4.4.5 Effluent COD concentration

Under normal activated sludge system operating conditions, where the sludge ages are in excess of 5 days (to ensure nitrification and biological nutrient removal), the nature of the influent organics in municipal wastewaters is such that the COD concentration in the effluent is inconsequential in the system design. The soluble readily biodegradable organics are completely utilized in a very short time ($< 2 \text{ h}$) and the particulate organics, whether biodegradable or unbiodegradable, are enmeshed in the sludge mass and settle out with the sludge in the secondary settling tanks. Consequently, *the effluent COD concentration comprises virtually wholly the soluble unbiodegradable organics (COD)* (from the influent) plus the COD of the sludge particles which

escape with the effluent due to imperfect operation of the secondary settling tank. Hence the filtered and unfiltered effluent COD concentration, COD_e , are given by:

$$COD_e = S_{U,e} \quad (\text{filtered mgCOD/l}) \quad (4.21a)$$

$$COD_e = S_{U,e} + f_{cv,OH} X_{VSS,e} \quad (\text{unfiltered mgCOD/l}) \quad (4.21b)$$

where:

$S_{U,e}$ unbiodegradable COD in the effluent = $S_{U,i}$
 $= f_{SU,CODi} COD_i$ (mgCOD/l)
 $X_{VSS,e}$ VSS in the effluent (mgVSS/l)
 $f_{cv,OH}$ COD/VSS ratio of the VSS
 (1.48 mgCOD/mgVSS)

In most cases the effluent VSS and TSS concentrations are too low to measure reliably with the VSS and TSS tests. Alternative methods for measuring low solid concentrations in the effluent have been developed; for the VSS, via the filtered and unfiltered COD concentrations, *i.e.* from Eq. 4.21, and for TSS, via the turbidity once a turbidity *versus* TSS concentration calibration curve for the activated sludge has been prepared (Wahlberg *et al.*, 1994).

$$X_{VSS,e} = (COD_{e(unfilt)} - COD_{e(filt)}) / f_{cv,OH} \quad (4.22)$$

4.4.6 The COD (or e^-) mass balance

In the activated sludge system, COD theoretically must be conserved so that at steady state the COD mass flow out of the system must equal to the COD mass flow into the system over a defined time interval. The COD (e^-) of the influent organics are (i) retained in the unbiodegradable particulate and soluble organics, (ii) transformed to OHO mass and therefore conserved in a different type of organic material, or (iii) passed to oxygen to form water. So in general the COD (or e^-) balance over the activated sludge system at steady state is given by Eq 4.23 where:

COD_e effluent total soluble COD concentration (mgCOD/l)
 X_{VSS} VSS concentration in biological reactor (mgVSS/l)
 O_c carbonaceous (for organic material degradation) oxygen utilization rate in reactor (mgO₂/l.h)

$$\begin{aligned} \left[\begin{array}{c} \text{Flux of COD}(e^-) \\ \text{output} \end{array} \right] &= \left[\begin{array}{c} \text{Flux of COD}(e^-) \\ \text{input} \end{array} \right] \\ \left[\begin{array}{c} \text{Flux of soluble} \\ \text{COD in} \\ \text{effluent} \end{array} \right] + \left[\begin{array}{c} \text{Flux of soluble} \\ \text{COD in waste} \\ \text{flow} \end{array} \right] &+ \left[\begin{array}{c} \text{Flux of particulate} \\ \text{COD in waste} \\ \text{flow} \end{array} \right] + \left[\begin{array}{c} \text{Flux of oxygen utilized} \\ \text{by OHOs for COD} \\ \text{breakdown} \end{array} \right] \\ &= \left[\begin{array}{c} \text{Flux of} \\ \text{COD input} \end{array} \right] \end{aligned}$$

$$\begin{aligned} Q_e COD_e + Q_w COD_e + Q_w X_{VSS} f_{cv,OH} + V_R O_c \\ = Q_i COD_i \end{aligned} \quad (4.23)$$

In Eq. 4.23, the first two terms represent the soluble organics that exit the system via effluent and waste flows, the third term the particulate organics that exit the system via the waste flow, and the fourth term the mass of oxygen utilized for biodegradable organic material breakdown by the OHOs. Noting that,

$$\begin{aligned} (Q_e + Q_w) COD_e &= Q_i COD_e = F COD_e \\ Q_w X_{VSS} &= V_R X_{VSS} / SRT = M X_{VSS} / SRT = F X_{VSS} \\ V_R O_c &= F O_c \\ Q_i COD_i &= F COD_i \end{aligned}$$

the general COD mass balance is given by,

$$F COD_e + f_{cv,OH} \cdot M X_{VSS} / SRT + F O_c = F COD_i \quad (4.24)$$

where:

$FCOD_e$ COD flux of soluble organics exiting system via effluent and waste flows (mgCOD/d)

$f_{cv,OHO} \cdot MX_{VSS}/SRT$

COD flux of particulate organics exiting system via waste flow (mgCOD/d)

FO_c flux of oxygen utilized by OHOs for biodegradable organic material degradation, all of those from the influent via the growth process and some of those from the OHO biomass via the endogenous respiration process (carbonaceous) (mgO₂/d)

Substituting Eqs 4.12 and 4.18 for MX_{VSS} and FO_c into Eq 4.24 gives the COD balance *versus* SRT, which is considered in Section 4.5.3 below (Figure 4.3).

The COD mass balance is a very powerful tool for checking (i) the data measured in experimental systems (Ekama *et al.*, 1986), (ii) the results calculated for design from the steady-state model and (iii) the results calculated by dynamic-simulation models. Application of the COD mass balance to nitrification and denitrification activated sludge systems is presented in Chapter 5.

4.4.7 Active fraction of the sludge

The active VSS mass MX_{OHOv} in the reactor is the live OHO mass which performs the biodegradation processes of the organic material. The other two organic solids masses, $MX_{E,OHOv}$ and MX_{Uv} are inactive and unbiodegradable and do not serve any function insofar as the biodegradation processes in the system are concerned. They are given different symbols because of their different origin, the MX_{Uv} is unbiodegradable particulate organics from the influent wastewater and the $MX_{E,OHOv}$ is unbiodegradable particulate organics produced in the reactor via the endogenous respiration process. The active OHO biomass fraction of the volatile solids in the reactor $f_{av,OHO}$ is given by,

$$f_{av,OHO} = \frac{MX_{OHOv}}{MX_{VSS}} \quad (\text{mg OHOVSS/mgVSS}) \quad (4.25)$$

Substituting eqs. 4.9 and 4.12 for MX_{OHOv} and MX_{VSS} and rearranging yields:

$$\frac{1}{f_{av,OHO}} = 1 + f_{XE,OHO} b_{OHO,T} SRT + \frac{f_{XU,CODi} (1 + b_{OHO,T} SRT)}{f_{cv,OHO} Y_{OHOv} (1 - f_{XU,CODi} - f_{SU,CODi})} \quad (4.26)$$

where:

$f_{av,OHO}$ active OHO fraction of the VSS mass

If the total suspended solids mass (TSS) is used as the basis for determining the active fraction, then the active fraction of the sludge mass with respect to the total suspended solids, $f_{at,OHO}$, is given by

$$f_{at,OHO} = f_{VT} f_{av,OHO} \quad (4.27)$$

where:

$f_{at,OHO}$ active OHO fraction of the TSS mass

f_{VT} VSS/TSS ratio of the activated sludge

The active fractions $f_{av,OHO}$ or $f_{at,OHO}$ give an indication of the 'stability' of the waste sludge, which is related to the remaining biodegradable organics in the sludge mass. The only biodegradable organics in the VSS mass are those of the OHOs which, in terms of the steady-state model, is 80% ($1 - f_{XE,OHO}$) of the OHO mass. Hence, the higher the active fraction, the greater the proportion of biodegradable organics remaining in the sludge mass and the greater the utilizable energy content remaining in the sludge mass (sections 4.11 and 4.13). For activated sludge to be stable, the remaining utilizable organics in it should be low so that it will not generate odours through further biological activity. Sludge used as a soil conditioner needs to be stable because its primary purpose is to provide nutrients and unbiodegradable organic content to the soil (Korentajer, 1991); unstable sludges applied to agricultural land lead to an undesirable high oxygen demand in the soil.

4.4.8 Steady-state design

The design equations set out above form the starting point for aerobic and anoxic-aerobic activated sludge system design. They apply to the simple single reactor completely mixed aerobic system and to the more complex multi-reactor anoxic-aerobic systems for biological nitrogen removal. When EBPR is included, the above equations *do not* give an accurate estimate of the VSS and TSS masses in the system. With EBPR, a second group of heterotrophic organisms, the PAOs need to be considered, which have different stoichiometric and kinetics constants producing more VSS and TSS mass per mass organics (COD) utilized. Incorporation of the PAOs in the steady-state model is discussed in Chapter 7.

For the more complex anoxic-aerobic systems, the above basic equations apply if the assumptions made in their derivation apply. Provided this is the case, the effects of nitrification (Chapter 5) and nitrification-denitrification (Chapter 5) and the associated oxygen demands can be formulated as additional equations to the basic equations above. The above ‘simplified’ approach is based on the assumption that the biodegradable organics are completely utilized. This has been established by the close correlation achieved between the mean response of the more complex anoxic aerobic systems predicted by the more complex general kinetic model (and validated experimentally), with that calculated by the above basic equations and the additional equations for nitrification and denitrification. The close correspondence between this simplified steady state model and the more complex general kinetic simulation models such as ASM1 is demonstrated by Sötemann *et al.* (2006) for aerobic and ND systems (including aerobic digestion). Indeed, these simple steady state models can form the basis for ‘hand’ calculations to (i) develop design input information for and (ii) check output results from the dynamic-simulation models (Ekama, 2009).

Other assumptions on which the steady state model is based are that (i) the mass of active OHOs seeded into the system with the influent is negligible in

comparison with that which grows in the reactor, (ii) there is no loss of solids in the effluent from the secondary settling tanks, (iii) water mass is conserved, (iv) a 100% COD balance is achieved, and (v) active OHO loss is modelled as endogenous respiration. It is important to understand these assumptions in the model. With regard to the assumption of complete utilization of biodegradable organics, if this is not the case, then the mass of sludge produced per day increases and the carbonaceous oxygen demand decreases below those predicted by the basic equations. The reason for these deviations lies in the kinetics of degradation of the slowly biodegradable particulate material; if, for example, the aerobic fraction of the sludge mass is too small (Chapter 5), the particulate biodegradable organics are only partially utilized and residual particulate biodegradable organics (X_s) accumulate in the system as additional VSS in the same way as the unbiodegradable particulate organics. Concomitantly, the carbonaceous oxygen demand is reduced because less biodegradable organics are utilized. Clearly, for such situations the simulation model results will deviate from steady-state ones. In fact, it would be wise to regard such deviations as a signal for possible incorrect output from the simulation model and start an investigation to find the cause for the deviation.

4.4.9 The steady-state design procedure

The calculation procedure to generate the design results required for a certain sludge age is as follows:

Select the wastewater characteristics $f_{XU,CODi}$ and $f_{SU,CODi}$ which are believed to best represent the unbiodegradable particulate and soluble COD fractions of the wastewater.

Then calculate:

- 1) $FX_{Uv,i}$ (Eq. 4.6) and $FX_{FSS,i}$ (Eq. 4.7)
- 2) $FCOD_i$ and/or $FCOD_{b,i}$ (Eq. 4.4 or 4.5)
- 3) Select the SRT
- 4) MX_{OHOv} (Eq. 4.9), $MX_{E,OHOv}$ (Eq. 4.10), MX_{Uv} (Eq. 4.11), MX_{VSS} (Eq. 4.12), MX_{FSS} (Eq. 4.14), MX_{TSS} (Eq. 4.15) or select f_{VT} , MX_{TSS} (Eq. 4.17)

- 5) FO_c (Eq. 4.18) and O_c (Eq. 4.19)
- 6) V_R (Eq. 4.20)
- 7) HRT_n , (Eq. 4.2)
- 8) COD_e (Eq. 4.21)

In this design procedure the input COD and its characteristics will be governed by the specific waste flow. The selection of values for the unbiodegradable soluble and particulate COD fractions is simple but not trivial. Each impacts the design in important areas. The unbiodegradable soluble COD fraction ($f_{SU,CODi}$) has a negligible influence on biological reactor design parameters, such as sludge production and oxygen demand, but has a marked influence on the effluent COD concentration (COD_e). In contrast, the unbiodegradable particulate COD fraction ($f_{XU,CODi}$) has no influence on the effluent COD concentration (COD_e) but has a marked influence on the specific sludge production rate (kgVSS produced/kgCOD load) and specific reactor volume ($m^3/kgCOD$ load per day). The higher the $f_{XU,CODi}$, the larger these values, and as the sludge age increases this influence of $f_{XU,CODi}$ becomes more marked. The system

parameter that requires selection is the sludge age. The sludge age selected will depend on the specific requirements from the wastewater treatment plant such as effluent quality, *i.e.* organic COD removal only, nitrification, N removal, biological P removal, and the envisioned sludge treatment facilities *i.e.* whether or not primary settling is included, the stability of the waste activated sludge etc. Specification of the sludge age, therefore, is an important design decision and requires special consideration (Section 4.13).

4.5 DESIGN EXAMPLE

The design procedure developed above for the fully aerobic activated sludge system is demonstrated with a numerical example. Assuming constant flow and load conditions, calculations are shown below how estimates of the system volume requirements, average daily carbonaceous oxygen demand, and daily sludge production are calculated for the treatment of the example raw and settled wastewaters (Table 4.2).

Table 4.2 Examples of raw and settled wastewater characteristics.

Parameter	Symbol	Unit	Raw	Settled
Flow	Q_i	ML/d	15	14.93
COD concentration	COD_i	mgCOD/l	750	450
Unbiodegradable particulate COD	$f_{XU,CODi}$		0.15	0.04
Unbiodegradable soluble COD	$f_{SU,CODi}$		0.07	0.12
Unbiodegradable soluble OrgN	$f_{SU,TKNi}$		0.03	0.035
TKN concentration	TKN_i	mgN/l	60	51
Total P concentration	P_i	mgP/l	15	12.75
TKN/COD ratio	$f_{TKNi/CODi}$	mgN/mgCOD	0.08	0.117
P/COD ratio	$f_{Pi/CODi}$	mgP/mgCOD	0.02	0.028
Temperature	T_{max}, T_{min}	°C	14-22	14-22
pH	-		7.5	7.5
H_2CO_3 alkalinity	$S_{Alk,i}$	mg/l as $CaCO_3$	250	250
Influent ISS	$X_{FSS,i}$	mgISS/l	47.8	9.5
VSS/TSS of activated sludge	f_{VT}	mgVSS/mgTSS	0.75	0.83

4.5.1 Temperature effects

From Table 4.1, the only kinetic constant in the steady state organic material (COD) degradation model for fully aerobic systems that is affected by temperature is the specific endogenous respiration rate b_{OHO} . This rate decreases by approximately 3% for every 1 °C decrease in temperature (*i.e.* $\theta_{b,\text{OHO}} = 1.029$). From Table 4.1, the rate at 14 °C is 0.202/d and at 22 °C is 0.254/d. The effect of the reduction in the rate with decrease in temperature is that at lower temperatures the daily sludge production is marginally increased and the average carbonaceous oxygen demand is marginally decreased. The differences in sludge production (kgVSS/d) and oxygen demand (kgO₂/d) are less than 5% for an 8 °C change in temperature from 14 to 22 °C. Consequently, the average carbonaceous oxygen demand should be calculated at the maximum temperature and the system volume and sludge production at the minimum temperature in order to find the maximum values of these parameters.

4.5.2 Calculations for organic material degradation

This design example demonstrates the effect of temperature and sludge age on (i) the mass of TSS sludge in the system (MX_{TSS} , kgTSS), (ii) the average daily carbonaceous oxygen demand (FO_c , kgO₂/d), (iii) the active fractions of the sludge with respect to VSS and TSS ($f_{\text{av},\text{OHO}}$ and $f_{\text{at},\text{OHO}}$) and (iv) the mass of TSS sludge wasted per day (FX_{TSS} , kgTSS/d). These four parameters are calculated for the example raw and settled wastewaters at 14 and 22 °C for sludge ages ranging from 3 to 30 days.

Mass of COD treated/d = $\text{FCOD}_i = Q_i \text{ COD}_i$ kgCOD/d

Mass of biodegradable COD treated/d = $\text{FCOD}_{b,i} = (1 - f_{\text{SU,COD}_i} - f_{\text{XU,COD}_i}) \text{ COD}_i$

Mass of unbiodegradable particulate organics flowing into the system as mgVSS /d = $\text{FX}_{\text{UV},i} = \text{FCOD}_i f_{\text{XU,COD}_i} / f_{\text{cv,UPO}}$

Hence for raw wastewater:

$$\text{FCOD}_i = 15 \text{ Ml/d} \cdot 750 \text{ mgCOD/l} = 11,250 \text{ kgCOD/d}$$

$$\text{FCOD}_{b,i} = (1 - 0.07 - 0.15) 11,250 = 8,775 \text{ kgCOD/d}$$

$$\text{FX}_{\text{UV},i} = 0.15 \cdot 11,250 / 1.48 = 1,140 \text{ kgVSS/d}$$

$$\text{FX}_{\text{FSS},i} = 15 \text{ Ml/d} \cdot 47.8 = 717 \text{ kgISS/d}$$

and for settled wastewater:

$$\text{FCOD}_i = 15 \text{ Ml/d} \cdot 450 \text{ mgCOD/l} = 6,750 \text{ kgCOD/d}$$

$$\text{FCOD}_{b,i} = (1 - 0.117 - 0.04) \cdot 6,750 = 5,690 \text{ kgCOD/d}$$

$$\text{FX}_{\text{UV},i} = 0.04 \cdot 6,750 / 1.48 = 182.4 \text{ kgVSS/d}$$

$$\text{FX}_{\text{FSS},i} = 15 \text{ Ml/d} \cdot 9.5 = 142.5 \text{ kgISS/d}$$

From eqs. 4.12 and 4.17, the masses of volatile (MX_{VSS}) and total (MX_{TSS}) settleable solids in the system is for the raw wastewater:

$$\text{MX}_{\text{VSS}} = 8,775 \frac{0.45 \text{SRT}}{(1 + b_{\text{OHO},T} \text{SRT})} \cdot (1 + 0.2b_{\text{OHO},T} \text{SRT}) + 1,140 \text{SRT} \quad (\text{kgVSS})$$

$$\text{MX}_{\text{FSS}} = 717 \text{ SRT} + 0.15 f_{\text{av},\text{OHO}} \text{MX}_{\text{VSS}} \quad (\text{kgFSS})$$

$$\text{MX}_{\text{TSS}} = \text{MX}_{\text{VSS}} / 0.75 \text{ or } \text{MX}_{\text{FSS}} + \text{MX}_{\text{VSS}} \quad (\text{kgTSS})$$

and for the settled wastewater:

$$\text{MX}_{\text{VSS}} = 5,690 \frac{0.45 \text{SRT}}{(1 + b_{\text{OHO},T} \text{SRT})} \cdot (1 + 0.2b_{\text{OHO},T} \text{SRT}) + 182.4 \text{SRT} \quad (\text{kgTSS})$$

$$\text{MX}_{\text{FSS}} = 142.5 \text{ SRT} + 0.15 f_{\text{av},\text{OHO}} \text{MX}_{\text{VSS}} \quad (\text{kgFSS})$$

$$\text{MX}_{\text{TSS}} = \text{MX}_{\text{VSS}} / 0.83 \text{ or } \text{MX}_{\text{FSS}} + \text{MX}_{\text{VSS}} \quad (\text{kgTSS})$$

From Eq. 4.18, the average daily carbonaceous oxygen demand is for the raw wastewater:

$$\text{FO}_c = 8,775 \cdot \left[(0.334) + 0.533 \frac{b_{\text{OHO},T} \text{SRT}}{(1 + b_{\text{OHO},T} \text{SRT})} \right] \quad (\text{kgO}_2/\text{d})$$

and for the settled wastewater:

$$FO_c = 5,690 \cdot \left[(0.334) + 0.533 \frac{b_{\text{OHO},T} \text{SRT}}{(1 + b_{\text{OHO},T} \text{SRT})} \right] \quad (\text{kgO}_2/\text{d})$$

From eqs. 4.26 and 4.27, the active fractions with respect to the VSS ($f_{\text{av},\text{OHO}}$) and TSS ($f_{\text{at},\text{OHO}}$) are for the raw wastewater:

$$f_{\text{av},\text{OHO}} = 1 / \left[\frac{1 + 0.2b_{\text{OHO},T} \text{SRT}}{+ 0.289(1 + b_{\text{OHO},T} \text{SRT})} \right]$$

and,

$$f_{\text{at},\text{OHO}} = 0.75 f_{\text{av},\text{OHO}}$$

and for the settled wastewater:

$$f_{\text{av},\text{OHO}} = 1 / \left[\frac{1 + 0.2b_{\text{OHO},T} \text{SRT}}{+ 0.142(1 + b_{\text{OHO},T} \text{SRT})} \right]$$

and,

$$f_{\text{at},\text{OHO}} = 0.83 f_{\text{av},\text{OHO}}$$

From the definition of sludge age (Eq. 4.1), the mass of VSS and TSS secondary sludge produced (or wasted) per day (FX_{TSS}) is for the raw wastewater:

$$\begin{aligned} FX_{\text{TSS}} &= Q_w X_{\text{TSS}} = MX_{\text{TSS}} / \text{SRT} \\ FX_{\text{TSS}} &= \frac{8,775}{0.75} \frac{0.45}{(1 + b_{\text{OHO},T} \text{SRT})} \\ &\cdot (1 + 0.2b_{\text{OHO},T} \text{SRT}) + \frac{1,140}{0.75} \end{aligned} \quad (\text{kgTSS}/\text{d})$$

and for the settled wastewater:

$$\begin{aligned} FX_{\text{TSS}} &= \frac{5,690}{0.83} \frac{0.45}{(1 + b_{\text{OHO},T} \text{SRT})} \\ &\cdot (1 + 0.2b_{\text{OHO},T} \text{SRT}) + \frac{142.8}{0.83} \end{aligned} \quad (\text{kgTSS}/\text{d})$$

The mass of VSS wasted/produced per day FX_{VSS} is simply f_{VT} times the mass of TSS wasted per day,

$$FX_{\text{VSS}} = f_{\text{VT}} \cdot FX_{\text{TSS}} \quad (\text{kgVSS}/\text{d})$$

Substituting the $b_{\text{OHO},T}$ value for 14 °C (*i.e.* 0.202/d) and 22 °C (*i.e.* 0.254/d) into the above equations allows MX_{VSS} , MX_{TSS} , FO_c , $f_{\text{av},\text{OHO}}$, $f_{\text{at},\text{OHO}}$, FX_{TSS} and FX_{VSS} to be calculated for sludge ages of 3 to 30 days. The results are shown plotted in Figure 4.3.

From these figures, the mass of sludge in the reactor (TSS or VSS), the average carbonaceous oxygen demand, the mass of TSS sludge produced per day (kgTSS/d) and the active fraction (with respect to MLSS or MLVSS) are only marginally affected by temperature and for these parameters; insofar as design is concerned, temperature effects are not really of much consequence. However, the influent waste type *i.e.* raw or settled sewage, has a significant effect. Raw sewage results in considerably more sludge in the system, a higher oxygen demand and a lower active fraction of the sludge than settled sewage. The difference in the effects of sewage type between raw and settled wastewater depends wholly on the efficiency of the PSTs. The differences apparent in Figure 4.3 arise from a 40% COD removal in the PST(s); the greater the COD removal efficiency, the greater the difference between the parameters for raw and settled sewage.

The results for 20 d sludge age are given in Table 4.3. From Eq. 4.20, for the same reactor TSS concentration, the system volume is proportional to the mass of sludge in it. Hence, for the same TSS concentration, the reactor volume treating settled sewage will be only 33% of that treating raw sewage at 20 days' sludge age. Also, the settled sewage plant requires only 63% of the oxygen the raw sewage plant requires. However, the active fraction with respect to TSS of the sludge in the settled sewage plant is 43%, too high for direct discharge to drying beds, whereas that from the raw wastewater plant is 23%. Clearly the choice of treating settled or raw wastewater requires weighing their advantages and disadvantages, *i.e.* for settled sewage a smaller reactor volume, lower

oxygen demand and reduced secondary sludge production, but having to deal with primary and secondary sludge and their stabilization, and for raw sewage, larger reactor volume, higher oxygen demand

and increased secondary sludge production, but having no primary sludge to deal with. These aspects are evaluated in further detail in Section 4.11.

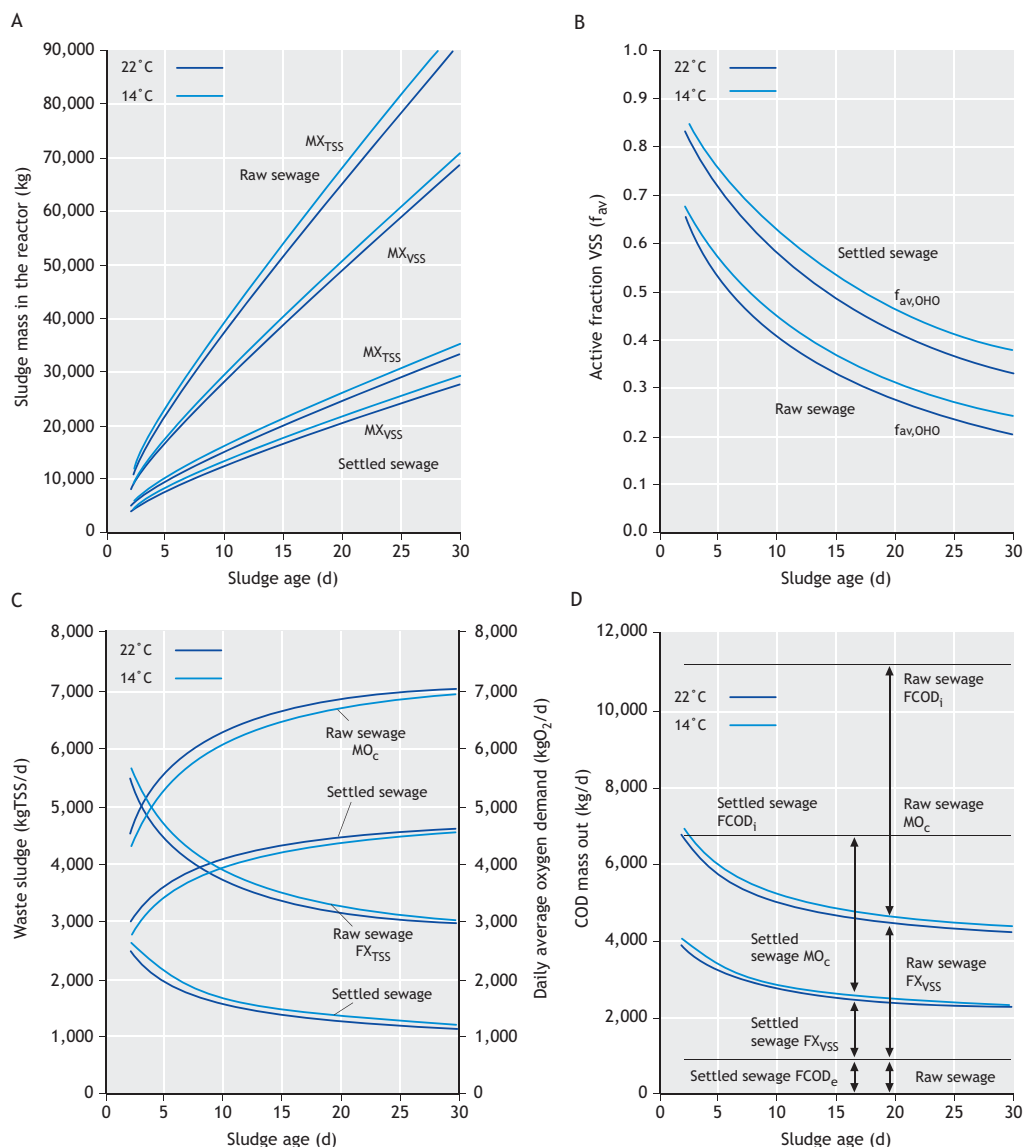


Figure 4.3 Mass of sludge MX_{TSS} (kgTSS) and MX_{VSS} (kgVSS) (A), active fractions of the sludge with respect to VSS ($f_{av,OHO}$) (B), average carbonaceous oxygen demand FO_c (kgO₂/d) and mass of TSS sludge produced per day (kgTSS/d) (C) and the COD mass balance (D) versus sludge age for the example raw and settled sewage at 14 °C and 22 °C. Stoichiometric and kinetic constants and wastewater characteristics are given in tables 4.2 and 4.3.

Table 4.3 Activated system design values for 20 days' sludge age for the example raw and settled wastewaters at 14 and 22°C.

System Parameter	Unit	Raw		Settled ¹	
Wastewater temperature	°C	14	22	14	22
Mass VSS, MX_{VSS}	kgVSS	51,122	48,982	21,918	20,537
Mass TSS, MX_{TSS}	kgTSS	68,162	65,309	26,408	24,743
Mass O_2 , FO_c	kg O_2 /d	6,679	6,837	4,313	4,415
Active fraction, $f_{av, OHO}$		0.306	0.265	0.461	0.408
Active fraction, $f_{at, OHO}$		0.23	0.199	0.383	0.339
Waste, FX_{VSS}	kgVSS/d	2,556	2,449	1,096	1,027
Waste, FX_{TSS}	kgTSS/d	3,408	3,265	1,320	1,237
Effluent COD, COD_e	mg/l	52.5	52.5	52.5	52.5
COD mass balance					
COD mass out	kgCOD/d				
Soluble in Q_e , $FCOD_e$		743	743	766	766
Oxygen utilized FO_c		6,679	6,837	4,313	4,415
Soluble COD in Q_w		45	17	45	17
COD of VSS in Q_w		3,783	3,625	1,622	1,520
Total COD out	kgCOD/d	11,249	11,249	6,718	6,718
Total COD in ¹	kgCOD/d	11,250	11,250	6,718	6,718
% COD mass balance		100	100	100	100

¹For settled sewage, based on an influent flow of 14.925 Ml/d to take account of the primary sludge flow of 75 m³/d (0.5% of influent ADWF)

4.5.3 The COD mass balance

Applying the COD mass balance Eq. 4.24 to the example raw and settled wastewaters yields:

- 1) soluble COD in effluent and waste flows,

$$(Q_e + Q_w = Q_i) : FCOD_e = COD_i Q_i = f_{SU, CODi} COD_i Q_i \quad (\text{kgCOD/d})$$

- 2) particulate COD (activated sludge) in waste flow,

$$(Q_w) : FCOD_{VSS} = \frac{f_{cv, OHO} MX_{VSS}}{SRT} \quad (\text{kgCOD/d})$$

where, MX_{VSS} is given by Eq. 4.12

- 3) carbonaceous oxygen utilized, FO_c , kg O_2 /d and given by Eq. 4.18.

- 4) mass COD entering system,

$$FCOD_i = COD_i Q_i \quad (\text{kgCOD/d})$$

Activated sludge systems operating at very long sludge ages, called extended aeration (e.g. 30 days, Table 4.4), allow the endogenous process to approach completion thereby providing not only sewage treatment in the activated sludge reactor but also a significant measure of aerobic stabilisation of the activated sludge to achieve a low active fraction so that the waste sludge can be discharged directly to drying beds without further treatment. Treating raw wastewater in an extended aeration system therefore obviates the need for sludge treatment, which takes place in the activated sludge reactor also, but this is at the expense of a very large activated sludge reactor and high oxygen utilization and energy consumption rates.

Table 4.4 Comparison of sludge production, stability (biodegradable COD remaining) and oxygen demand treating the example raw and settled wastewaters at long and short sludge ages.

Parameter	Units	Raw	Settled
Temperature	°C	14	14
Sludge age	d	30	8
Activated sludge concentration	mgTSS/l	4,000	4,000
Reactor volume	m ³	23,769	3,544
Oxygen demand	kgO ₂ /d	6,944	3,758
Primary sludge TSS	kgTSS/d	0	3,335
Primary sludge VSS	kgVSS/d	0	2,468
Primary sludge COD	kgCOD/d	0	4,531
Biodegradable COD remaining	%	0	68.5
Secondary sludge TSS	kgTSS/d	3,169	1,772
Secondary sludge VSS	kgVSS/d	2,377	1,471
Secondary sludge COD	kgCOD/d	3,518	2,177
Active fraction with regard to VSS	kgOHOVSS/kgVSS	0.235	0.662
Biodegradable COD remaining	%	18.8	53
Total sludge TSS	kgTSS/d	3,169	5,107
Total sludge VSS	kgVSS/d	2,377	3,939
Total sludge COD	kgCOD/d	3,518	6,708
Biodegradable COD remaining	%	18.8	63.5

In contrast, treating settled sewage at a very short sludge age (high rate, *e.g.* 8 days), the activated sludge system results in a very small activated sludge reactor and a low oxygen utilization and energy consumption but produces a primary sludge and a very active waste activated sludge, both of which need aerobic or anaerobic treatment for stabilisation to reduce the remaining biodegradable organics.

With anaerobic digestion these sludges can produce a significant amount of methane gas and offset some (or all) of the power consumption for aeration. The remaining biodegradable COD in the WAS, as a proportion of the influent COD flux entering the activated sludge system, for increasing SRT is shown in Figure 4.4 for the example raw and settled wastewater.

It can be seen that the remaining biodegradable COD in the WAS, which is essentially the biodegradable part of the OHO biomass, rapidly decreases as SRT increases, with the result that less and less energy can be extracted from it with anaerobic digestion at the cost of generating a digester dewatering liquor high in ammonia and phosphate (Ekama, 2017). At very short sludge ages, the focus of the activated sludge system therefore is sewage treatment only, with sludge treatment (stabilisation) taking place in separate dedicated aerobic or anaerobic systems for this with possible energy and nutrient recovery. Irrespective of the approach adopted (extended aeration or high rate) for the particular sewage treatment plant, the COD mass must balance over the entire plant, not only over the activated sludge system but over the sludge treatment systems also.

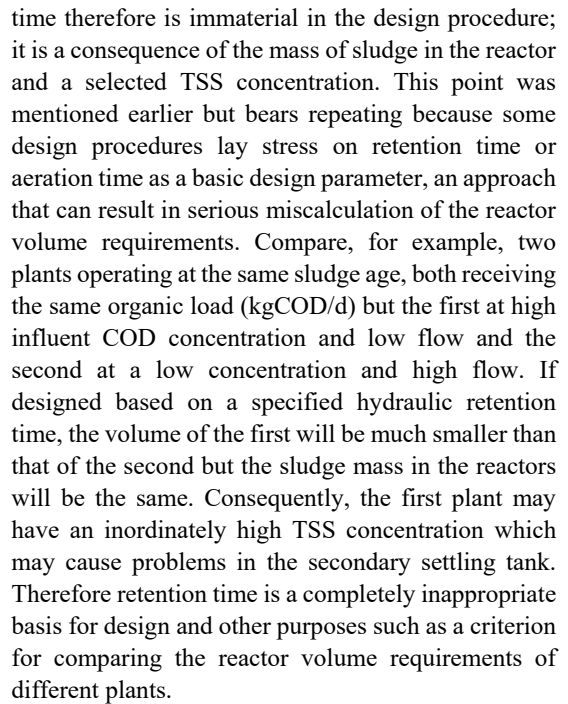


Figure 4.5 shows the reactor volume requirements *versus* sludge for the example raw and settled wastewaters obtained from eqs. 4.4 to 4.17 and 4.20. The reactor volume requirements may also be determined from the equivalent COD load per capita or person equivalent (PE), also shown in Figure 4.5 for a raw wastewater COD load of 0.10 kgCOD/PE.d. Hence when treating the example raw wastewater at a sludge age of 20 days and a TSS concentration of 4 kgTSS/m³, a reactor volume of 145 l per PE is required or 1.45 m³/kgCOD applied per day to the treatment plant. The comparative reactor volume requirements for settled wastewater per kg COD load per day on the treatment plant also is shown in Figure 4.5 taking due consideration of the COD fraction removed by primary sedimentation (40% for the example of settled wastewater).

From Figure 4.5, treating settled wastewater at a sludge age of 20 days and reactor TSS concentration of 4 kgTSS/m³ requires a reactor volume of 0.55 m³/kg raw wastewater COD load per day on the wastewater treatment plant or 55 l per PE. Comparing

Once the mass of sludge in the reactor is derived from a specified sludge age and organic COD mass load per day, the reactor volume is determined by ‘diluting’ this mass of sludge to a specified TSS concentration (X_{TSS}). From the volume, the nominal hydraulic retention time, or aeration time for fully aerobic systems, is fixed (by Eq. 4.2). Hydraulic retention

the reactor volume requirements treating raw or settled sewage, it can be seen that a significant reduction in reactor volume can be obtained by means of primary sedimentation: 62% for the example raw and settled sewage at 20 days' sludge age.

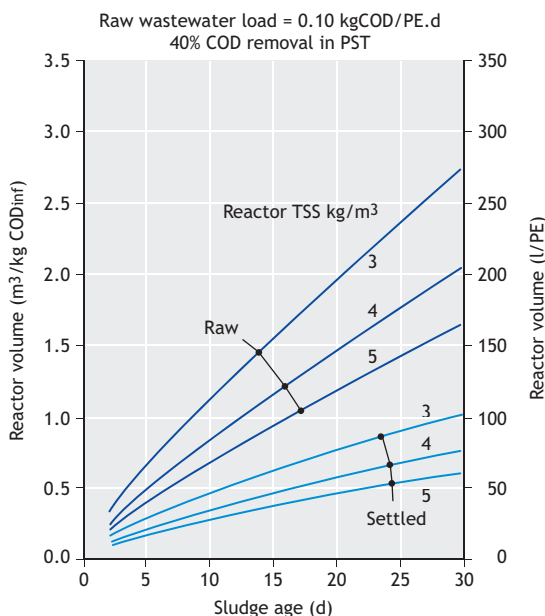


Figure 4.5 Reactor volume requirements in m^3/kg COD raw wastewater load per day versus sludge age at different average reactor TSS concentrations for raw and settled wastewater (assuming 40% COD removal by primary sedimentation). Reactor volume requirements in l/capita or l/person equivalent (PE) is also given on the right-hand vertical axis based on a raw wastewater COD contribution of $0.10 \text{ kg COD}/\text{person}$ equivalent.

4.7 DETERMINATION OF REACTOR TSS CONCENTRATION

The choice of the reactor concentration can be done empirically from past experience with similar wastewaters or selected from design guidelines such as those from Metcalf and Eddy (2014), *e.g.* for conventional systems (with primary sedimentation) $1,500$ to $3,000 \text{ mgTSS}/\text{l}$ or extended aeration (without primary sedimentation) $3,000$ to $6,000 \text{ mgTSS}/\text{l}$.

Differences in the reactor TSS concentration for raw and settled wastewaters arise because (i) the wastewater flow per kg COD load on the reactor for raw wastewater is significantly greater than that for settled wastewaters and (ii) sludge settleability in conventional systems can be poorer than with extended aeration systems: in a survey of 45 full-scale activated sludge plants in the Netherlands, Stofkoper and Trentelman (1982) found significantly higher DSVIs in settled wastewater systems than in raw wastewater systems (Ekama and Marais, 1986).

The effect of sewage strength and sludge settleability, as well as other factors such as the peak wet weather (PWWF) to average dry weather flow (ADWF) ratio (or peak flow factor $f_q = \text{PWWF}/\text{ADWF}$), wastewater and activated sludge characteristics ($f_{\text{XU},\text{CODi}}$, $f_{\text{SU},\text{CODi}}$, f_{VT}) and construction costs, can all be taken into account by determining the reactor concentration from a construction cost minimization analysis (Hörler, 1969; Dick, 1976; Riddell *et al.*, 1983; Pincince *et al.*, 1995). In such an analysis, the construction cost of the reactor(s) and the secondary settling tank(s) (SSTs) are determined as functions of the reactor TSS concentration. The reactor concentration at which the combined construction cost of the reactor(s) and the SST(s) is at a minimum is the design reactor concentration.

4.7.1 Reactor cost

For selected wastewater and activated sludge characteristics ($f_{\text{XU},\text{CODi}}$, $f_{\text{SU},\text{CODi}}$, f_{VT}), sludge age and organic COD load on the reactor ($\text{FCOD}_{\text{i,Reactor}}$), the mass of TSS in the reactor (MX_{TSS}) can be determined from eqs. 4.15 or 4.17 and remains constant, *e.g.* for the example raw sewage at 20 days sludge age and 14°C , $\text{MX}_{\text{TSS}} = 68,162 \text{ kgTSS}$. The reactor volume (V_R) in a single module (V_M) as a function of the reactor TSS concentration X_{TSS} is found from Eq. 4.20, *viz.*

$$V_M = \text{MX}_{\text{TSS}} / X_{\text{TSS}} = \frac{68,162}{N_{\text{AS}} X_{\text{TSS}}} \quad (\text{m}^3)$$

where:

X_{TSS} reactor concentration (kgTSS/m^3)

N_{AS} number of AS modules with one reactor

per module in order to keep the reactor volume of one module within a maximum reactor volume limit, *i.e.* $V_R = N_{AS} \cdot V_M$.

To estimate the cost of the reactor from the volume, empirical functions relating the construction cost of the reactor to the volume are required. Such functions take the form

$$\text{Reactor costs} = N_{AS} C_{br} (V_R)^{P_{br}} \quad (4.28)$$

where:

C_{br} , P_{br} constants for a particular reactor design.

4.7.2 Secondary settling tank cost

On the basis of the flux theory, Ekama *et al.* (1997) show that provided the underflow recycle ratio s is above the critical minimum value, the surface area of the secondary settling tanks (A_{SST}) is a function of only the reactor (or feed) solids concentration (X_{TSS}) and the sludge settleability. If the reactor concentration increases or the sludge settleability deteriorates, the required surface area for the secondary settling tanks (SSTs) gets larger. Therefore, as the biological reactor gets smaller with increasing X_{TSS} , so A_{SST} gets larger. Hence, the construction cost of the SSTs increases with increases in X_{TSS} .

To determine the surface area of the SSTs, two parameters need to be specified for the design, *viz.* (i) the sludge settleability and (ii) the peak flow factor f_q (= PWWF/ADWF ratio). The idealized 1D flux theory requires the sludge settleability to be specified in terms of the V_0 and r_{hin} values in the zone settling velocity (V_s , m/h) *versus* solids concentration (X_{TSS} , kgTSS/m³) relationship *i.e.*

$$V_s = V_0 \exp(-r_{hin} X_{TSS}).$$

Values for V_0 and r_{hin} are not readily available but relationships between different simpler sludge settleability parameters such as the sludge volume index (SVI), stirred specific volume index (SSVI) and diluted sludge volume index (DSVI) have been proposed by various authors (see Ekama *et al.*, 1997).

These relationships allow calculation of the flux V_0 and r_{hin} values from the SVI, SSVI or DSVI sludge settleability indices. However, there is considerable variation in these relationships and selection for a particular activated sludge plant needs very careful consideration. For this example, the relationships developed by Ekama and Marais (1986) are accepted:

$$SSVI_{3.5} = 0.67 DSVI \quad (\text{ml/g}) \quad (4.29a)$$

$$V_0 / r_{hin} = 67.9 \exp(-0.016 SSVI_{3.5}) \quad (\text{kgTSS/m}^2 \cdot \text{h}) \quad (4.29b)$$

$$r_{hin} = 0.88 - 0.393 \log(V_0 / r_{hin}) \quad (\text{m}^3/\text{kgTSS}) \quad (4.29c)$$

$$V_0 = (V_0 / r_{hin}) r_{hin} \quad (\text{m/h}) \quad (4.29d)$$

From the 1D idealized flux theory, the maximum permissible overflow rate at PWWF ($q_{i,PWWF}$) is given by,

$$q_{i,PWWF} = 0.8 V_s \text{ at } X_{TSS} = V_0 \exp(-r_{hin} X_{TSS}) \quad (\text{m/h}) \quad (4.30a)$$

where:

$q_{i,PWWF}$ overflow rate at PWWF (m/h)
0.8 flux factor

$$q_{i,PWWF} = Q_{i,PWWF} / A_{SST} = f_q Q_{i,ADWF} / A_{SST} \quad (\text{m/h}) \quad (4.30b)$$

From a calibration of the 1D idealized flux theory SST design procedure against full-scale SST performance data, Ekama and Marais (2004) showed that the maximum permissible solids loading rate (SLR, kgTSS/m²·h) should only be 80% of that estimated by the 1D idealized flux theory and is the reason for the 0.8 in Eq 4.30. This reduction appears to be a consequence of the significant deviation of the hydrodynamics in real SSTs compared with that assumed in the 1D idealized flux theory such as horizontal flows of liquid and solids, turbulence, short circuiting and density currents (Ekama *et al.*, 1997).

Taking the 25% (1/0.80) reduction into account, the surface area of the SST(s), A_{SST} , in terms of X_{TSS} is given by,

$$A_{SST} = \frac{1,000 f_q Q_{i,ADWF} / 24}{0.8 V_0 \exp(-r_{hin} X_{TSS}) N_{AS} N_{SST}} \quad (m^2) \quad (4.31)$$

where:

$Q_{i,ADWF}$ average dry weather flow (ml/d)
 N_{SST} number of SSTs per AS module and governed by the maximum diameter of one SST.

Functions for the construction cost function in terms of diameter (ϕ , m) for circular SSTs of given depth may take the form:

$$SSTcost = N_{AS} N_{SST} C_{sst}(\phi)^{P_{sst}} \quad (4.32)$$

where:

C_{sst} , P_{sst} constants for a particular design.

4.7.3 Total cost

The total cost of the reactor-SST system is the sum of the reactor and SST costs. Qualitative results for the example raw and settled wastewaters are given in Figure 4.6, ignoring that the reactor volume and SST

diameter may have upper and lower size restrictions. For full-scale plants, the reactor and/or SST may need to be split into two or more equal-sized modules to bring the volume and diameter within the limit ranges. From cost minimization analyses such as that above, generally it will be found that the range of reactor concentration for minimum construction cost (i) is higher for higher influent wastewater strengths (BOD₅, COD), (ii) is higher for longer sludge ages, and (iii) is higher for raw wastewater than settled wastewater at the same strength, because these three changes all increase the size of the biological reactor relative to that of the settling tank, (iv) is lower for higher peak flow factors (f_q), and (v) is lower for poorer settling sludges because these two changes all increase the size of the settling tank relative to that of the biological reactor. A universal optimum therefore cannot be specified. In countries with low wastewater strengths and short sludge-age plants (e.g. North America), the reactor concentration will tend to be low (2,000-3,000 mgTSS/l) and in countries with high wastewater strengths and long sludge-age plants (e.g. South Africa), the reactor concentration will tend to be high (4,000-6,000 mgTSS/l) as the example wastewaters demonstrate.

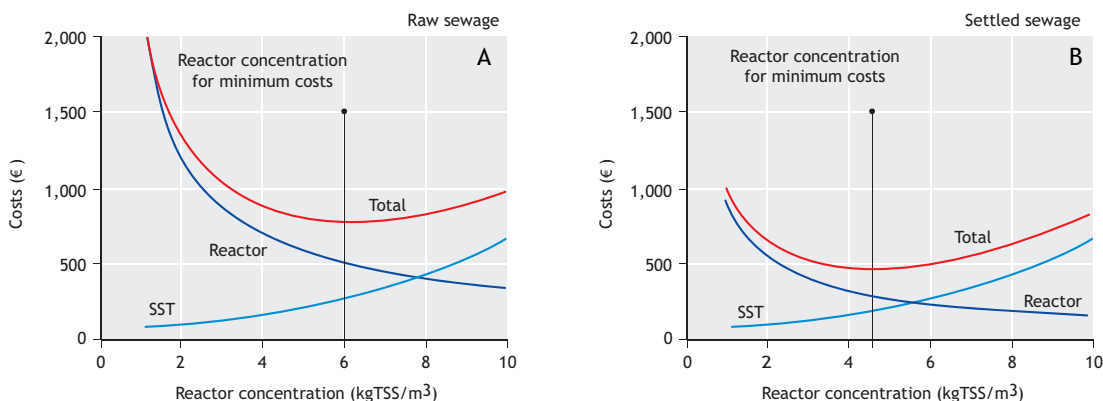


Figure 4.6 Reactor, secondary settling tank (SST) and total construction costs to estimate the reactor concentration for minimum cost for the example raw (A) and settled (B) wastewaters in single-reactor and SST units.

4.8 CARBONACEOUS OXYGEN DEMAND

4.8.1 Steady-state (daily average) conditions

The mean daily carbonaceous oxygen demand per kg COD load on the reactor ($FO_c/FCOD_{i,Reactor}$) is calculated from Eq. 4.18. For sludge ages longer than 15 days the increase in $FO_c/FCOD_{i,Reactor}$ is small with further increase in sludge age for both raw and settled wastewater. The $FO_c/FCOD_{i,Reactor}$ for raw and settled wastewater is usually within 10% of each other, with the demand for settled wastewater being the higher value. This is because compared with raw wastewater, a higher percentage of the total organics (COD) in settled wastewater is biodegradable. For the example wastewaters at 20 days' sludge age, the $FO_c/FCOD_{i,Reactor}$ is 0.604 kgO₂/kgCOD for raw wastewater and 0.653 kgO₂/kgCOD for settled wastewater. Although there is only a small difference in $FO_c/FCOD_{i,Reactor}$ between raw and settled wastewaters, there is a large difference in the oxygen demand per kgCOD load on the plant (Figure 4.3C). For settled wastewater, this is given by $0.653 \cdot (1-0.40)$ for 40% COD removal in PSTs. This gives 0.38 kgO₂/kgCOD load on the plant. For the raw wastewater it would remain 0.604 kgO₂/kgCOD load on the treatment plant, making the settled wastewater oxygen demand 37% lower than that for the raw wastewater. Clearly primary sedimentation leads to significant aeration energy savings; because primary settling tanks remove about 30 to 50% of the raw influent COD, the carbonaceous oxygen demand for settled wastewater generally will be about 30 to 50% lower than that for raw wastewater.

The carbonaceous oxygen demand is the oxygen demand for the oxidation of the influent organics (COD) and the associated OHO endogenous process only. In N removal systems, oxygen is also required for nitrification, which is the biological oxidation of ammonia to nitrate by autotrophic nitrifiers. However, with denitrification, which is the biological reduction of nitrate to nitrogen gas by facultative heterotrophic organisms, some of the biodegradable organics are utilized with nitrate as electron acceptor, for which

oxygen is then not required. Thus denitrification leads to a reduction in the oxygen demand. The total oxygen demand for a N-removal system therefore is the sum of the carbonaceous and nitrification oxygen demands, less that saved by denitrification. The procedures for calculating the oxygen demand for nitrification and the oxygen saved by denitrification are discussed in Chapter 5. The equations given here for calculating the carbonaceous oxygen demand are based on the assumption that all the biodegradable organics are utilized with oxygen as the electron acceptor, *i.e.* for fully aerobic systems.

4.8.2 Daily cyclic (dynamic) conditions

Owing to the daily cyclic nature of the organic (COD) load on the reactor, the carbonaceous oxygen demand will vary concomitantly over the day. The TKN load on the reactor also varies over the day in an approximately similar fashion as the organic load. Generally, the COD and TKN loads on the reactor increase in the morning due to increases in both flow and COD and TKN concentration reaching a peak at around noon. Thereafter, the COD and TKN loads decrease reaching a minimum during the night-time hours of 2:00 to 4:00 due to decreases in both flow and COD and TKN concentration. The peak to average and minimum average load ratios and the time of day these occur depend on the catchment that the particular treatment plant serves, such as size of population, layout of the catchment and industrial activity. Generally, the smaller the catchment, the lower the flow and COD and TKN loads but the greater the peak to average flow and load ratios then the lower the minimum-to-average flow and load ratios. Because the TKN load and its variation over the day and the nitrification process have a profound influence on the daily average and peak total oxygen demands, empirical methods to estimate the peak oxygen demand from the average for fully aerobic nitrifying systems are discussed in Chapter 5. Fully aerobic activated sludge systems with sludge ages longer than 3 days are likely to nitrify at temperatures $> 14^\circ\text{C}$. Moreover, a sludge age of 3 days is around the limit of validity for the steady state activated sludge model because at sludge ages lower than this

the assumption that all the biodegradable organics are utilized is not valid. So there is little merit in developing empirical methods for estimating the peak oxygen demand for fully aerobic systems without nitrification.

4.9 DAILY SLUDGE PRODUCTION

The mass of sludge produced per day by the activated sludge system is equal to the mass of sludge wasted per day from it via the waste flow and is called waste activated sludge (WAS) or secondary sludge. From the definition of sludge age (see Eq. 4.1), the mass of sludge TSS produced per day FX_{TSS} is given by the mass of sludge in the system MX_{TSS} divided by the sludge age *i.e.*:

$$FX_{TSS} = MX_{TSS} / SRT \quad (\text{mgTSS/d}) \quad (4.33)$$

Substituting eqs. 4.12 and 4.17 for MX_{TSS} and simplifying, yields the sludge produced per day per mg COD load on the biological reactor, *i.e.*

$$\frac{FX_{TSS}}{FCOD_i} = \frac{1}{f_{VT}} \left[\frac{(1 - f_{SU,CODi} - f_{XU,CODi}) Y_{OHov}}{(1 + b_{OH0,T} SRT)} \cdot (1 + f_{XE,OH0} b_{OH0,T} SRT) + \frac{f_{XU,CODi}}{f_{cv,UPO}} \right] \quad (\text{mgTSS/d per mgCOD/d}) \quad (4.34)$$

A plot of the daily total sludge mass (TSS) produced per unit COD load on the biological reactor (Eq. 4.34) *versus* sludge age is shown in Figure 4.7 for the example raw and settled wastewaters.

It can be seen that the mass of sludge produced in the activated sludge system (per unit COD load on the biological reactor) decreases as the sludge age increases for both raw and settled wastewater but the rate of decrease is negligible at sludge ages longer than approximately 20 days. Treating settled wastewater results in lower secondary sludge production per unit COD load on the biological reactor than treating raw wastewater. This is because the unbiodegradable particulate COD fraction

($f_{XU,CODi}$) and inorganic content ($X_{FSS,i}/COD_i$) in settled wastewater are significantly lower than that in raw wastewater.

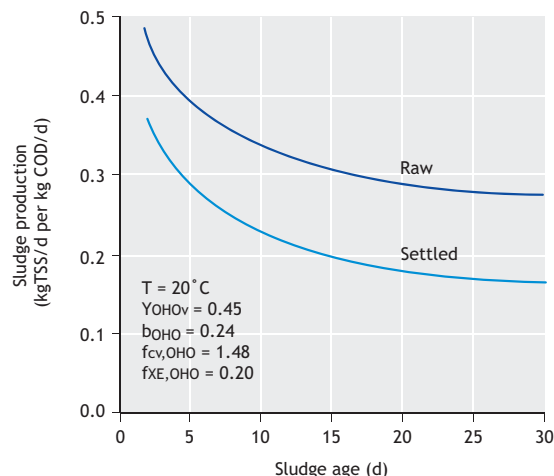


Figure 4.7 Daily sludge production in kgVSS/d and kgTSS/d per kgCOD load per day on the biological reactor for the example raw and settled wastewaters at 14 °C.

Temperature effects on secondary sludge production are small: sludge production at 14 °C is about 5% greater than at 22 °C, a difference which is completely masked by the uncertainty in the estimates of the wastewater characteristic $f_{XU,CODi}$ and the VSS/TSS ratio (f_{VT}) of the sludge if the influent ISS concentration ($X_{FSS,i}$) is not measured.

Although the secondary sludge production treating settled wastewater is lower than that treating raw wastewater, the *total* sludge mass treating settled wastewater is higher because the total sludge production includes both the primary and secondary sludges; at plants treating raw wastewater, only secondary sludge is produced.

In the system treating raw wastewater, the primary sludge is in effect treated in the activated sludge reactor itself. From the COD balance, the more oxygen that is utilized in the system, the lower the sludge production and the lower the active fraction of

the sludge (Figure 4.3B,C). Therefore, because the carbonaceous oxygen demand is much higher when treating raw wastewater, the overall sludge production is much lower compared with settled wastewater.

Generalizing the above observations and taking into account the active fraction of the waste sludge as an indication of the remaining biodegradable organics in the waste sludge, there are two extremes in approach to designing wastewater treatment plants with activated sludge for biological treatment:

- (i) treating settled wastewater at a short sludge age (say 8 days): this results in a very small activated sludge system with low oxygen demand and a high sludge production with high energy content, *i.e.* high remaining biodegradable organics in both the primary and secondary (waste activated) sludges requiring further stabilization treatment before disposal, or,
- (ii) treating raw wastewater at a long sludge age (say 30 days): this results in a very large activated sludge system with high oxygen demand and a low sludge production with a low energy content *i.e.* no primary sludge and low remaining biodegradable organics (low active fraction) in the secondary sludge, not requiring further stabilization treatment before disposal.

The daily production of secondary and primary sludges is the mass of sludge that needs to be treated and disposed of by downstream sludge handling methods. Sludge treatment and disposal, for biological nutrient removal (BNR) systems in particular, should not be seen as separate from the design of the activated sludge system. In fact, all unit operations of the treatment plant from raw wastewater pumping to ultimate disposal of the sludge, should be viewed as an integrated system where the design of one unit operation depends on the unit operations before it, and decisions on its design may affect the design of unit operations following it. For example, while approach 1 above provides an opportunity for low energy consumption and high energy generation by anaerobic digestion, the impact of the ammonia and phosphate in the digested sludge dewatering liquor on the

activated sludge reactor needs to be considered unless N and P removal from the dewatering liquor is included in the sludge treatment scheme (Ekama, 2017). N removal from dewatering liquor is considered in Chapter 5.

4.10 FOOD-TO-MICRO-ORGANISM (F/M) RATIO AND LOAD FACTOR

The food-to-microorganism (F/M) ratio and load factor (LF) have been used for decades to size activated sludge reactors. The F/M seeks to quantify the flux of biodegradable organics (food, COD, BOD₅) on the activated sludge microorganisms (m) or OHO biomass. Because there was no alternative, it was accepted that the VSS, being organic, satisfactorily quantified this OHO biomass. Without expressly referring to biodegradable organics and micro-organisms, the load factor carries the same idea as the F/M by expressing as a ratio the flux organic load (kgCOD/d or kgBOD₅/d) on the sludge mass in the reactor (kgVSS or kgTSS). However, both these ratios are seriously flawed from a modelling perspective. To mention a few (Wentzel *et al.*, 2003): (i) while the BOD₅ is an approximation of the biodegradable organics, it measures neither the electron-donating capacity of the biodegradable organics nor the unbiodegradable particulate organics (UPO), which accumulate as VSS in the activated sludge reactor. (ii) The BOD₅ is not mass conservative and so cannot be used to make a BOD mass balance around the activated sludge system. (iii) Although the COD is not the 'food' because it includes the unbiodegradable organics, most importantly it is mass conservative and so a COD mass balance can be made around the activated sludge system. (iv) As the AS model in the previous sections shows, the VSS (and less so the TSS) are not measures of the OHO biomass. The F/M and LF ratios are entirely empirical and for the same activated sludge system, have different numerical values depending on the units (COD, BOD₅, VSS, TSS) selected to express them. Also, the ratios vary with the wastewater characteristics (particularly the UPO fraction) and temperature. In fact, as long ago as 1976, Marais and Ekama show that the F/M or LF add no additional

information to the AS system that sludge age (SRT) does not provide because by selecting an F/M or LF, one is implicitly selecting a system SRT. To make matters worse, selecting the same F/M or LF expressed in the same units for AS systems treating different raw or settled wastewater, yields different SRTs depending on the wastewater characteristics. Yet F/M or LF-based empirical design procedures continue to be taught in environmental engineering programs and used in consulting practices as the basis for activated sludge system design. Because SRT forms the basis for modeling and simulation of AS systems, the link between F/M or LF and SRT is given below. This will demonstrate that the F/M and LF are implicitly embedded in and can be completely replaced by the much better SRT as the fundamental basis for activated sludge system sizing and modelling. This will allow the results from the empirical F/M or LF-based design procedures to be compared with the ASM1-aligned steady-state activated sludge model presented in the above sections. Using the TSS and COD for the reactor suspended solids and influent organics concentrations, and considering fully aerobic or nitrification denitrification (ND) systems only, then from eqs. 4.12 and 4.14,

$$A_{ND,TSS} = \frac{MX_{TSS}}{FCOD_i} = (1 - f_{SU,CODi} - f_{XU,CODi}) \cdot \frac{Y_{OHO}SRT}{(1 + b_{OHO,T}SRT)} (1 + f_{XE,OHO}b_{OHO,T}SRT + f_{FSS,OHO}) + \left[\frac{f_{XU,CODi}}{f_{cv,UPO}} + \frac{X_{FSS,i}}{COD_i} \right] SRT \quad (4.35)$$

where:

$A_{ND,TSS}$ or $MX_{TSS}/FCOD_i$ is the inverse of the F/M or LF in units kgTSS/(kgCOD.d).

Note that $A_{ND,TSS}$ is units-specific and AS system type-specific. It is different for ND and ND-enhanced biological P removal (EBPR) systems and varies with the magnitude of biological P removal (Ramphao *et al.*, 2006, see Chapter 7, Section 7.8). Hence the A value as expressed in Eq 4.35 is valid for ND systems

only and has units kgTSS/(kgCOD/d). Dividing Eq 4.35 by the SRT gives the specific sludge production ($SP_{TSS,COD}$) as kgTSS produced/d per kgCOD load/d, which is the same as Eq. 4.34, *i.e.*,

$$\frac{1}{\left(\frac{F}{M}\right)_{COD,TSS}} = \frac{1}{LF_{COD,TSS}} = SP_{TSS,COD} R_s \quad (4.36a)$$

$$= \frac{MX_{TSS}}{FCOD_i} = \frac{V_R X_{TSS}}{Q_i COD_i} = A_{ND,TSS}$$

$$\left(\frac{F}{M}\right)_{COD,TSS} = LF_{COD,TSS} = \frac{1}{A_{ND,TSS}} \quad (\text{kgCOD/kgTSS.d}) \quad (4.36b)$$

where:

- MX_{TSS} mass of TSS in reactor (kgTSS)
= $V_R \cdot X_{TSS,ave}$
- V_R volume of reactor (m^3)
- $X_{TSS,ave}$ average TSS concentration in reactor (kgTSS/ m^3)
- $SP_{COD,TSS}$ specific sludge production factor in kgTSS/d/kgCOD.d, Metcalf & Eddy (2014).

Equation 4.36 shows that choosing an F/M or LF (as some of the obsolete design procedures do) is the same as choosing in a single lumped parameter the SRT and wastewater characteristics UPO and USO COD fractions ($f_{XU,CODi}$, $f_{SU,CODi}$) and influent ISS concentration ($X_{FSS,i}$). Therefore, if the wastewater characteristics $f_{XU,CODi}$, $f_{SU,CODi}$ and $X_{FSS,i}$ are known, then the SRT implicit in the selected F/M can be calculated from eqs 4.35 and 4.36. This is shown in figures 4.8a and 4.8b, in which the F/M ratio in kgBOD₅/(kgTSS.d) is plotted *versus* SRT for the raw and settled wastewater characteristics in Table 4.2 and COD/BOD₅ ratios for raw and settled wastewater of 2.2 and 1.8, respectively (Marais and Ekama, 1976). The right-hand side of figures 4.8A and 4.8B show the ranges of F/M or LF ratios for AS systems for carbon (COD) removal only, C removal with ND and extended aeration, which usually means raw wastewater treatment at long SRT to obviate waste-activated sludge (WAS) stabilization.

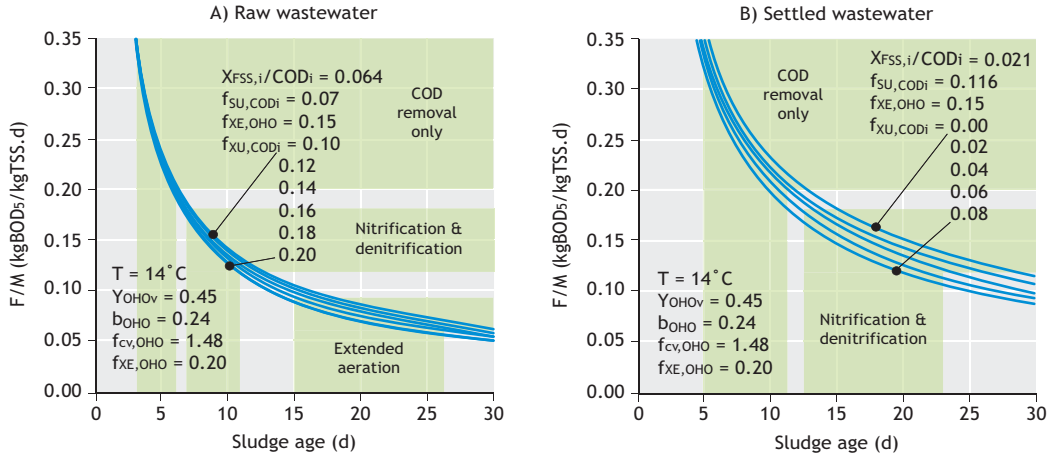


Figure 4.8 F/M ratio versus sludge age for different unbiodegradable particulate COD fractions (UPO , $f_{\text{XU,COD}i}$) for raw wastewater with a COD/BOD₅ ratio of 2.2 (A) and settled wastewater with a COD/BOD₅ ratio of 1.8 (B).

The F/M in terms of kgVSS in reactor per kgCOD load/d is obtained from Eq 4.35 by setting to zero the terms relating to ISS, *i.e.* $X_{\text{FSS},i}=0$ and $f_{\text{FSS,OHO}}=0$, *viz.*

$$\begin{aligned} \frac{1}{\left(\frac{F}{M}\right)_{\text{COD,VSS}}} &= \text{SP}_{\text{VSS,COD}} \text{SRT} = A_{\text{ND,VSS}} = \frac{MX_{\text{VSS}}}{\text{FCOD}_i} \\ &= (1 - f_{\text{SU,COD}i} - f_{\text{XU,COD}i}) \frac{Y_{\text{OHO}} \text{SRT}}{(1 + b_{\text{OHO},T} \text{SRT})} \\ &\quad \cdot (1 + f_{\text{XE,OHO}} b_{\text{OHO},T} \text{SRT}) + \left[\frac{f_{\text{XU,COD}i}}{f_{\text{cv,UPO}}} \right] \text{SRT} \end{aligned} \quad (4.37)$$

where:

$\text{SP}_{\text{COD,VSS}}$ the specific sludge production factor in kgVSS/d/kgCOD/d,
 $A_{\text{ND,VSS}}$ mass of VSS in the reactor per kgCOD load per day for fully aerobic and ND systems, which is a function of SRT and the wastewater characteristics.

4.11 CAPACITY ESTIMATION OF AS SYSTEMS

For a given biological reactor volume (V_R , m³) and SST surface area (A_{SST} , m²) the average dry weather flow ($Q_{i,\text{ADWF}}$) capacity in Ml/d can be calculated for a particular wastewater with known characteristics ($f_{\text{XU,COD}i}$, $f_{\text{SU,COD}i}$ and $X_{\text{FSS},i}$). With the required system SRT known, *e.g.* the balanced SRT of a Modified Ludzack-Ettinger (MLE) ND system (Chapter 5, Section 5.9.3.2), the mass of TSS in the reactor (MX_{TSS} , kgTSS) per kgCOD/d load applied to the reactor [$A_{\text{ND,TSS}}$ kgTSS/(kgCOD/d)] can be determined from Eq. 4.35,

$$MX_{\text{TSS}} = V_R X_{\text{TSS,ave}} = A_{\text{ND,TSS}} Q_{i,\text{ADWF}} \text{COD}_i \quad (\text{kgTSS}) \quad (4.38)$$

where $X_{\text{TSS,ave}}$ is the volume-weighted average reactor TSS concentration (kgTSS/m³).

In some BNR system reactors the TSS concentration is not the same in each zone of the reactor, *e.g.* in a UCT EBPR system, the TSS concentration in the anaerobic compartment ($X_{\text{TSS,AN}}$) is lower than that in the aerobic compartment

($X_{TSS,OX}$). This makes the volume-weighted average reactor TSS concentration ($X_{TSS,ave}=MX_{TSS}/V_R$) lower than the aerobic compartment TSS concentration ($X_{TSS,OX}$) fed to the SST (Parco *et al.*, 2018).

Rearranging Eq 4.38 yields,

$$Q_{i,ADWF} = \frac{V_R X_{TSS,ave}}{A_{ND,TSS} COD_i} \quad (ML/d) \quad (4.39)$$

Equation 4.39 has two unknowns, $Q_{i,ADWF}$ and $X_{TSS,ave}$, so one more fact is required to determine $Q_{i,ADWF}$. This is obtained from the overflow rate on the SST. The overflow rate at PWWF must not be greater than the flux rating (0.8) times settling velocity (V_s , m/h) of the sludge at the SST feed or aerobic zone TSS concentration as stated by Eq 4.30. Rearranging Eq 4.31 to make $Q_{i,ADWF}$ the subject yields,

$$Q_{i,ADWF} = \frac{24A_{SST} 0.8V_0}{1,000 f_q e^{-r_{hin} X_{TSS,OX}}} \quad (ML/d) \quad (4.40)$$

where V_0 and r_{hin} are in units m/h and $m^3/kgTSS$ and are given by Eq 4.29.

Setting Eq 4.40 equal to Eq 4.39 requires an equation linking $X_{TSS,ave}$ and $X_{TSS,OX}$. This is obtained from the BNR system configuration. For fully aerobic, MLE and 4 stage Bardenpho ND systems without step feeding, the TSS concentrations in all compartments of the reactor are the same. Hence $X_{TSS,ave}=X_{TSS,OX}$ and setting Eq 4.39 = Eq 4.40 yields,

$$X_{TSS,ave} = B_{ND} e^{-r_{hin} X_{TSS,ave}} \quad (kgTSS/m^3) \quad (4.41a)$$

where

$$B_{ND} = \frac{A_{ND,TSS} COD_i A_{SST} 0.8V_0 24}{f_q V_R 1,000} \quad (kgTSS/m^3) \quad (4.41b)$$

where $A_{ND,TSS}$, COD_i , A_{SST} , V_0 , 24 and V_R have units ($kgTSS.d$)/ $kgCOD$, $mgCOD/l$, m^2 , m/h and h/d and 0.8, f_q and 1,000 are dimensionless.

With $X_{TSS,ave}$ known, $Q_{i,ADWF}$ is given in ML/d either by Eq 4.39 or Eq 4.40.

The equation linking $X_{TSS,ave}$ and $X_{TSS,OX}$ for a particular BNR system configuration can be derived from TSS mass balances around each compartment of the reactor and SST, for which the waste flow rate (Q_w) can be set to zero to simplify the derivation. Equations for several different ND and NDEBPR systems with SST (and membranes) are given (Parco *et al.*, 2018) in chapters 5 and 6, respectively.

For a sludge settleability DSVI of 120 $ml/gTSS$, a PWWF/ADWF ratio (f_q) of 2.5, the mass of TSS (MX_{TSS}) in the reactor treating raw and settled WW at 20d SRT and 14 °C from Table 4.3 and a reactor TSS concentration that minimizes the total cost of the reactor and SST of 5.22 and 4.02 $kgTSS/m^3$ from Section 4.7 above, yields reactor volumes and SST areas for the raw and settled WW systems listed in Table 4.5.

Table 4.5 Optimum AS reactor volume and SST area for minimum total cost giving maximum ADWF capacity.

Wastewater type	Raw WW	Settled WW
Mass TSS in reactor (MX_{TSS} , Table 4.3)	68,162	26,408
Reactor TSS content for minimum total cost ($kgTSS/m^3$)	5.22	4.02
Sludge settleability (DSVI, $ml/gTSS$)	120	120
Reactor volume (V_R , m^3)	13,049	6,577
SST area (A_{SST} , m^2)	1,972 (50.1)	1,248 (39.9)
SST volume at 3.5 m avg. depth (m^3)	6,903	4,367
Total AS-SST volume (m^3)	19,952	10,944

If the SST are 3.5 m deep, then the reactor is 65% and 60%, respectively of the total AS-SST volume for the raw and settled WW AS-SST systems. Accepting

these AS systems are each in a single module with one SST, then from eqs. 4.38 to 4.41, the ADWF capacity of the raw and settled WW systems for deteriorating sludge settleability (increasing DSVI from 60 to 180 ml/gTSS) are shown in Figure 4.9. As expected, the ADWF capacity decreases qualitatively as sludge settleability deteriorates, but eqs 4.38 to 4.41 quantify this decrease.

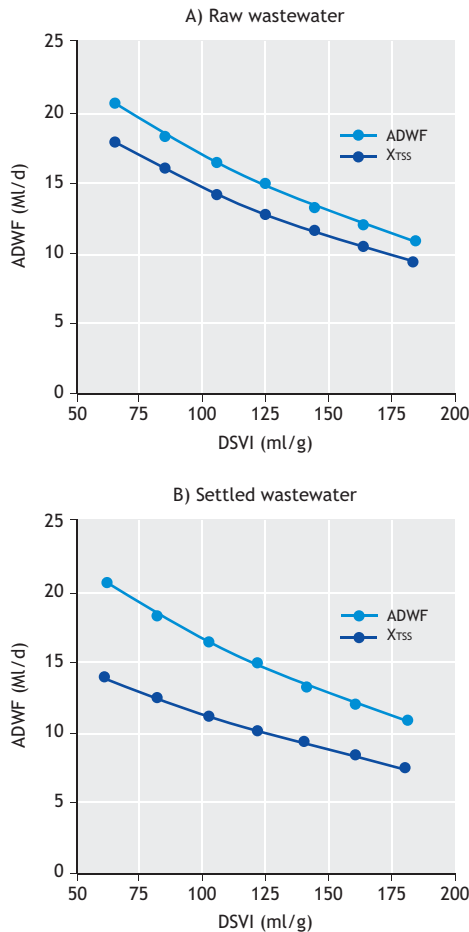


Figure 4.9 ADWF treatment capacity versus DSVI for the raw wastewater (WW) (A) and settled WW (B) AS systems given in Table 4.5.

If the SST are assigned an average depth of say 3.5 m then the total volume of the raw and settled WW AS-SST systems are 19,952 and 10,944 m³, respectively, making the reactor volume 65% and 60% of the total system volume (Table 4.5). Figure 4.10 shows the ADWF capacity of the total AS-SST system volume for increasing the % volume assigned to the reactor and therefore decreasing the % volume assigned to the SST.

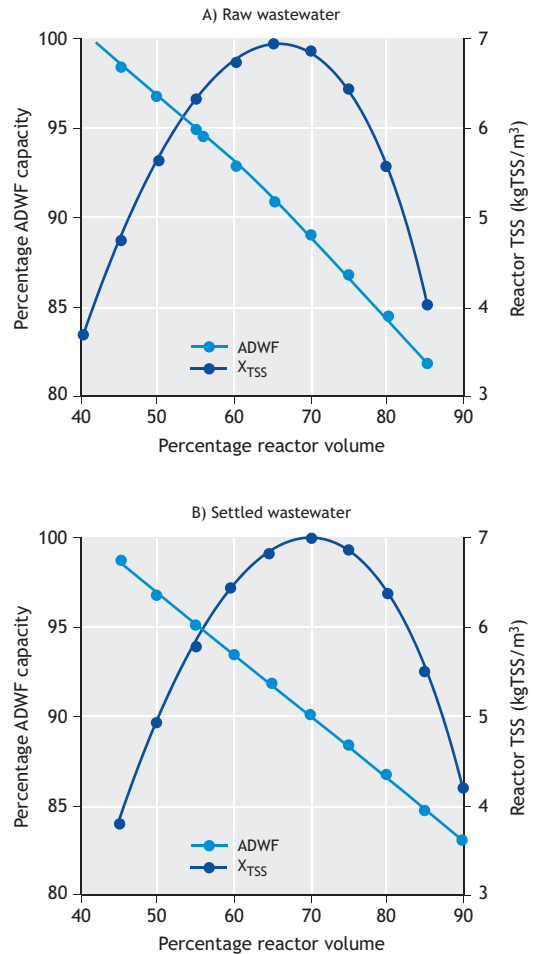


Figure 4.10 ADWF capacity, as a % of the maximum, and reactor TSS concentration for increasing % volume of the total AS-SST system volume assigned to the reactor for raw WW (A) and settled WW (B).

As the reactor occupies a larger and larger % of the total system volume, the reactor TSS concentration decreases and the SST area gets smaller. Figure 4.10 shows that for both raw and settled WW, within $\pm 13\%$, *i.e.* over a range of 26% of the 65% and 60% for the total volume for the reactor volume, the ADWF capacity is above 95% of the maximum capacity. The reason for this is because the total cost of the AS-SST system is reasonably flat over quite a wide TSS concentration range (see Figure 4.6). This indicates that obtaining a precise reactor TSS concentration to determine the reactor volume and SST area is not so critically important; a TSS concentration reasonably close (within approximately ± 0.5 kgTSS/m³) to the optimum that minimizes the total system cost is sufficient. From Figure 4.6, it can be seen that as the cost of the SST increases relative to that of the reactor, this tolerance range for the reactor TSS concentration narrows.

For the reactor volume and SST areas in Table 4.5, the ADWF capacity for raw and settled WW for increasing SRT is shown in figures 4.11a and b. At very short SRT (< 5 d), the ADWF capacity is almost double that at long SRT (> 20). Hence selecting the shortest possible SRT ensures the highest treatment capacity for a given AS reactor volume and SST area. While for organics removal only, there are not well-defined quantified criteria to select the shortest possible system SRT, for ND systems there are. It will be shown in Chapter 5 that the SRT of ND systems is based on sustaining the nitrifiers in the AS system. The autotrophic nitrifiers are the slowest growing functional organisms in the NDEBPR system and therefore the SRT, and hence the reactor volume is governed by their maximum specific growth rate. Calculation of the shortest possible (minimum) SRT for ND (and NDEBPR) systems is therefore possible and is presented in chapters 5 and 6.

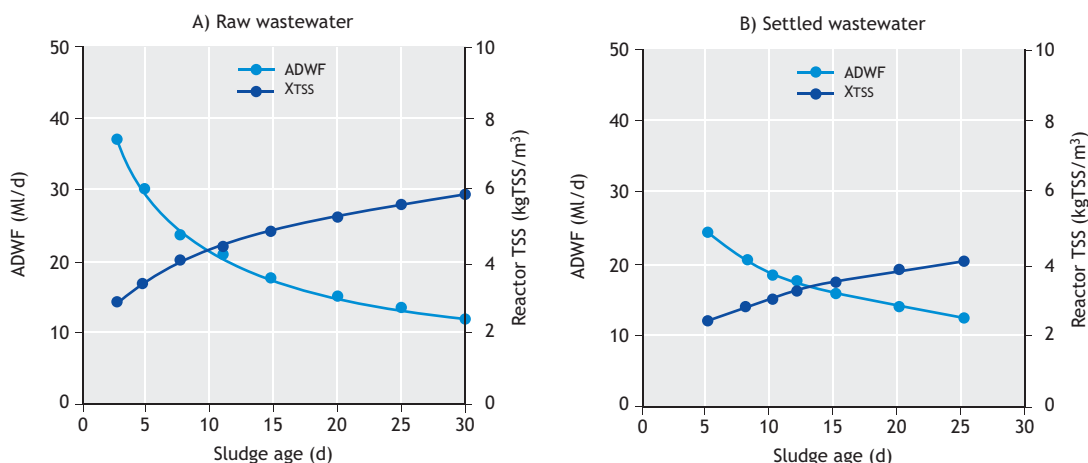


Figure 4.11 ADWF capacity in ML/d and reactor TSS concentration for increasing SRT for the reactor volume and SST area given in Table 4.5 for raw WW (A) and settled WW (B).

4.12 SYSTEM DESIGN AND CONTROL

The parameter of fundamental importance in the design and control of the activated sludge system is the sludge age, which governs the mass of sludge to be wasted daily from the system. The sludge age can and should completely replace the food-to-microorganism ratio (F/M, kg BOD or COD load per day per kg MLSS or MLVSS in the reactor) or equivalently the load factor (LF) as a reference and control parameter, *in particular if nitrification is required*. The sludge age can be fixed by a simple control procedure if the system is appropriately designed. This control procedure is simpler and operationally more practical and reliable than procedures based on the F/M or LF, which basically seek to control the mass of sludge in the system by controlling the reactor MLSS concentration at some specified value.

4.12.1 System sludge mass control

By far the most common activated-sludge-system control procedure involves keeping the sludge MLSS concentration in the reactor at some specified value. At best this sludge concentration is specified by design or at worst, established from operational experience of the plant behavior, which is usually the concentration that can be contained in the system by the secondary settling tanks (SSTs). This approach does not control sludge age, only the mass of sludge in the system. In fact, in some instances, it may not even be the sludge mass that is controlled via the reactor concentration, but the settled volume at 30 min (SV_{30}) in the 1 l measuring cylinder. If the SV_{30} is greater than say 450 ml/l, then sludge is wasted until it reaches this value again. This approach was developed to obviate the need for measuring the reactor sludge concentration and with it, the sludge concentration in the reactor varied with the sludge settleability (SVI). This approach was acceptable before nitrification became obligatory and at least ensured that the sludge could be contained in the system while maintaining a low effluent suspended solids (ESS) concentration. However, with this method there is no control of the F/M, LF, sludge

mass, reactor concentration or sludge age, a situation which is completely untenable when nitrification is required. While nitrification is a simple process to cater for in the design (just make the sludge age long enough and provide sufficient oxygen), it imposes a completely different control regime on the operation of the system. It requires the sludge age to be controlled at a fixed value.

If the F/M or LF is controlled, then in order to keep these parameters within the desired limits, not only does the reactor concentration need to be measured regularly, but also the daily BOD₅ (or COD) load. This requires extensive sampling and testing of the influent BOD₅ (or COD) concentration and flow pattern over the day to determine the daily COD (or BOD) mass load. Controlling the sludge age requires measuring the reactor MLSS concentration and the mass of sludge wasted per day. Usually the waste sludge is abstracted from the SST underflow to benefit from its thickening function. However, the sludge concentration of the underflow varies considerably over the day with the daily cyclic flow through the plant (figures 4.12 and 4.13).

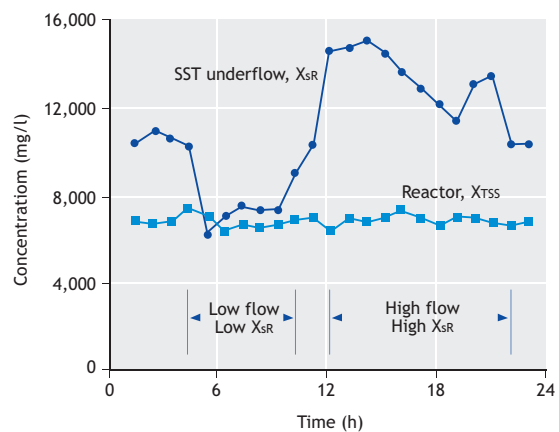


Figure 4.12 Experimental data from a full-scale activated sludge plant illustrating the virtually constant reactor concentration compared with the widely varying SST underflow concentration over the day (data: Nicholls, 1975).

Therefore, in order to know the sludge mass wasted via the underflow, it is necessary to measure the underflow concentration, waste flow rate and duration each time sludge is wasted. Thus to know the LF or sludge age, intensive testing of the influent and/or reactor and underflow concentrations are required. This may be manageable at large plants where the technical capacity is adequate, but at small plants, both the LF and sludge age are usually not known. As a consequence, nitrification is sporadic, partial or stops altogether during periods of poor sludge settleability, which results in high sludge wastage and hence short sludge age.

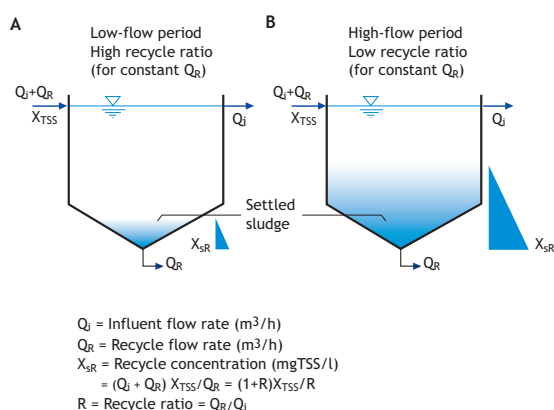


Figure 4.13 Increased sludge accumulation and higher underflow sludge concentration at high influent flow periods (B) than at low influent flow periods (A) at constant recycle flow rate.

Even if the reactor concentration were accurately controlled with modern control equipment such as automated wasting and on-line reactor concentration measurement, this still does not control the sludge age. With reactor concentration control at the same value throughout the year and a stable organic load on the plant (zero urban development), the sludge age decreases during winter because sludge production per kg COD load increases with decrease in temperature due to the lower endogenous respiration rate. While the decrease is relatively small, decreasing the sludge age is nevertheless the opposite of what

should be done to the system in winter to keep the ammonia concentration low. This is particularly relevant to plants operated at sludge ages close to the minimum for nitrification (Chapter 5), which is common practice in developed countries to squeeze as much capacity as possible out of the plant because space for extensions is limited. If the reactor concentration is controlled and the organic load on the plant progressively increases, which is usually the case in developing countries where urban growth is often constrained by plant capacity, the sludge age decreases progressively with time (Figure 4.14). Inevitably, on one cold winter's day, nitrification will have stopped. The operators' response is that an industrial discharge has inhibited the nitrifiers and so they do not change their operating procedures and wait for nitrification to return, which it does when the wastewater warms again in the summer.

When nitrification is required, not only should sludge age be controlled, but also the SST can no longer serve the dual purpose of clarifier and waste activated sludge (WAS) thickener. To obtain high WAS concentrations, the underflow recycle ratio must be low ($< 0.25:1$), which results in long sludge residence times in the SST (Figure 4.8).

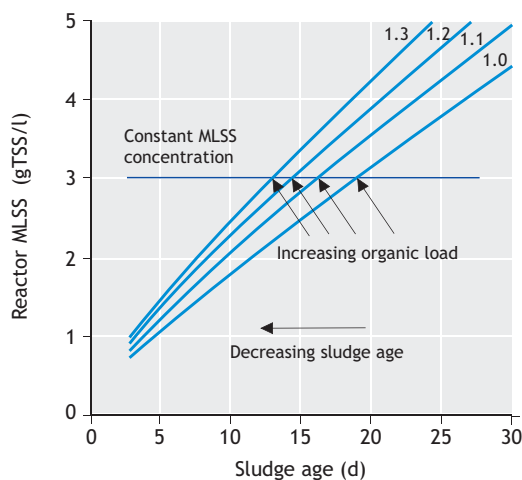


Figure 4.14 Decreasing SRT with increasing organics load when controlling reactor MLSS concentration.

The long sludge residence time stimulates denitrification in the SSTs causing floating (or rising) sludge on the SST surface, in particular in summer when wastewater temperature is high ($> 20^{\circ}\text{C}$). In fact, in the tropics, the climatic region of most developing countries, it may not be possible to operate an activated sludge system that does not nitrify even at very short sludge ages. So the problem of rising sludge due to denitrification can take place in plants even where nitrification is not a requirement. This happened at the Brasilia wastewater treatment plant

(Figure 4.15A and B), which had a low return sludge ratio (0.25:1): it nitrified even at 3 days sludge age and suffered from floating sludge all the time. If the sludge age was reduced to stop nitrification, the COD removal deteriorated below an acceptable level. So once nitrification takes place, whether intentionally by design or unavoidably, one must cater for denitrification in appropriate reactor (anoxic) zones and increase the underflow recycle ratio ($\sim 1:1$) to minimize rising sludge in the SSTs due to denitrification.



Figure 4.15A One of two wastewater treatment plants in Brasilia, Brasil (photo: R. Brummer).



Figure 4.15B Rising sludge due to denitrification in the SST at one of the wastewater treatment plants in Brasilia, Brasil (photo: M. Henze).

Clearly, once nitrification takes place, whether as a requirement for N removal or unavoidably due to system conditions, one is forced to abandon using the SSTs as WAS thickeners. If one has to thicken WAS in a separate unit, whether from the underflow or reactor, one may as well waste sludge directly from the reactor and derive the significant operational benefit of hydraulic control of sludge age. It is simple, requires very little testing and establishes the sludge age almost exactly. It results in stable all-year-round nitrification and is strongly recommended for activated sludge systems where nitrification is required, even where sophisticated reactor concentration control measures can be applied.

4.12.2 Hydraulic control of sludge age

Hydraulic control of sludge age was first proposed and implemented in a generalized form by Garrett in 1958, and is based on a method of 'modified wastewater aeration' implemented by Setter *et al.* (1945). If a sludge age of 10 days is specified, $1/10^{\text{th}}$ of the reactor volume is wasted daily, and if 20 d, $1/20^{\text{th}}$ is wasted daily, *i.e.* $Q_w = V_R / \text{SRT}$ (Eq. 4.1). For plants with low levels of technical support, a satellite settling tank or a dewatering drying bed completely independent of the SSTs can be provided to which the daily WAS flow from the reactor is discharged; for plants with a higher level of technical support a dissolved air flotation unit would be best (Bratby, 1978), which also minimizes P release from EBPR sludges (Pitman, 1999). The supernatant is returned to the reactor and the thickened sludge is pumped to the sludge treatment/disposal part of the plant. This procedure establishes very closely the desired sludge age because the mixed liquor concentration does not change significantly over the day (Figure 4.12).

An important aspect about hydraulic control of the sludge age is that *irrespective of the flow through the plant*, if a fixed fraction of the volume of the reactor is wasted every day, the sludge age is fixed. If the COD mass load per day on the plant remains constant, the sludge concentration will automatically remain constant. If the COD mass load increases, the sludge concentration will increase automatically, to maintain

the same sludge age. Thus, by monitoring the reactor concentration and its changes at a fixed sludge age, an indirect measure is obtained of the long-term changes in COD load on the plant. With time the reactor concentration may increase, indicating that the organic load on the plant is increasing (Figure 4.16). Hydraulic control of sludge age is very easy for the operator: (s)he just needs to check that the flume/pipe is not blocked and running at the correct flow rate, and the reactor MLSS concentration does not even have to be measured very often.

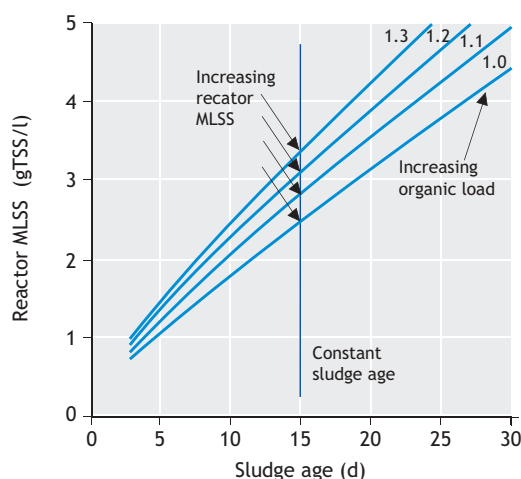


Figure 4.16 Constant SRT but increasing reactor MLSS concentration with hydraulic control of SRT.

By means of the hydraulic control procedure, the sludge age may be changed by simply changing the volume wasted per day. If say, the sludge age is reduced from 25 days to 20 days by hydraulic control, the full effect of the change will become apparent only after about half a sludge age. Thus, the biomass has an opportunity to adapt gradually to the change in F/M and LF. Hydraulic control of sludge age is particularly relevant to plants with sludge ages longer than about 5 days because for these plants the mass of sludge contained in the SSTs is a relatively small fraction of the total mass of sludge in the system. At sludge ages shorter than 5 days the mass of sludge in the SSTs can

become appreciable with respect to the total mass of sludge in the system, particularly when the sludge settleability becomes poor ($DSVI > 150$ ml/g). When the mass of sludge in the SSTs is significant, hydraulic control will have to take this into account and accuracy of the control will require additional testing.

Hydraulic control of sludge age devolves a greater responsibility onto the designer and removes responsibility from the plant operator; often operator ingenuity had to work around design inadequacies by force fitting the biological processes into the designed constraints to achieve the best effluent quality. It becomes essential that the designer calculates the sludge mass more exactly, to provide sufficient reactor volume under the design organic load to allow for the required reactor concentration at the specified sludge age. Also, the settling tank surface area, underflow recycle ratio and aeration capacity must be accurately sized for the particular wastewater and sludge age of the system. If these aspects are catered for adequately, then with hydraulic control of the sludge age, plant control is simplified and, for small-scale plants, may even do away with the requirements for solids and SVI tests except at long intervals. Hydraulic control of sludge age makes parameters such as LF and F/M redundant and introduces an entirely different attitude to system control. It is eminently practical and establishes the desired sludge age to ensure all-year-round nitrification. When nitrification is a requirement, sludge age control becomes a requirement, and then hydraulic control of sludge age is the easiest and most practical way to do this. Moreover, with hydraulic control of sludge age the mode of failure of the plant is completely different than with solids mass control. With the solids mass control the plant fails due to nitrification stopping and a high effluent ammonia concentration, a non-visible dissolved constituent which is also difficult to remove by other means. With sludge age control, the plant fails more obviously: sludge over the secondary settling tank effluent weirs. At plants managed with low levels of technical capacity, this is more likely to prompt remedial action.

4.13 SELECTION OF SLUDGE AGE

Selection of the sludge age is the most fundamental and important decision in the design of an activated sludge system. The sludge age selected for a plant depends on many factors, some of which are listed in Table 4.6 such as stability of the system, sludge settleability, whether or not the waste sludge needs to be suitable for direct discharge to drying beds, and most important of all, the quality of effluent required *i.e.* is COD removal only acceptable, must the effluent be nitrified, is nitrogen and phosphorus removal required. Several of the factors have already been discussed earlier and will not be repeated here. Only a few clarifying and additional comments on Table 4.6 will be made below.

4.13.1 Short sludge ages (1 to 5 days)

4.13.1.1 Conventional plants

These plants are operated in the conventional configuration *i.e.* a semi plug-flow configuration, but modified systems such as contact stabilization, step aeration, step feed and others are also implemented. Short sludge-age plants were extensively used in Europe and North America before N (and P) removal became requirements. Their main objective is COD removal only, for which sludge ages of 1 to 3 days are sufficient. BOD_5 or COD reductions range from 75 to 90%. The removal achieved depends on the wastewater characteristics, the operation of the plant in particular the management of the transfer of the sludge between the reactor and SSTs, and the efficiency of the SSTs. Because predatory activity of protozoan organisms on the free swimming bacteria is limited at short sludge ages, the non-settling component (or dispersion) of the activated sludge flocs is high which causes turbidity and high effluent COD (Chao and Keinath, 1979; Parker *et al.*, 1971).

Table 4.6 Some important considerations in the selection of sludge age for the activated sludge system.

Sludge age	Short (1 to 5 days)	Intermediate (10 to 15 days)	Long (>20 days)
Types	High rate, step feed, aerated lagoons, contact stabilization, pure oxygen	Similar to high rate but with nitrification and sometimes denitrification. BNR systems	Extended aeration Orbal Carousel BNR systems
Objectives	COD removal only	COD removal Nitrification Biological N removal and/or biological P removal	COD removal Biological N removal Biological P removal
Effluent quality	Low COD High ammonia High phosphate Variable	Low COD Low ammonia Low nitrate High/low phosphate Relatively stable	Low COD Low ammonia Low nitrate Low phosphate Usually stable
Primary settling	Generally included	Usually included	Usually excluded
Activated sludge quality	High sludge production Very active Stabilization required	Medium sludge production Quite active Stabilization required	Low sludge production Inactive No stabilization required
Oxygen demand	Very low	High due to nitrification	Very high due to nitrification and long sludge age
Reactor volume	Very small	Medium to large	Very large
Sludge settleability	Generally good, but bulking by non-low F/M filaments such as <i>S. natans</i> , 1701, <i>Thiothrix</i> possible.	Good at low sludge age and high aerobic mass fractions; but generally poor due to low F/M filament growth such as <i>M. parvicella</i> .	Can be good with high aerobic mass fractions, but generally poor due to low F/M filament growth particularly <i>M. parvicella</i>
Operation	Very complex due to AS system variability and primary and secondary sludge treatment.	Very complex with BNR and 1 st and 2 nd sludge treatment	Simple if without primary and secondary sludge treatment, but BNR system is complex.
Advantages	Low capital costs Energy self-sufficient with anaerobic digestion	Good biological N (and P) removal at relatively low capital cost.	Good biological N (and P) removal No primary and stable secondary sludge Low sludge handling costs
Disadvantages	High operation costs effluent quality variation	Complex and expensive sludge handling costs	Large reactor, high oxygen demand, high capital cost

It is accepted in Table 4.6 that short-sludge-age plants would not normally nitrify. For temperate and high latitude regions, where wastewater temperatures are generally below 20 °C, this would be the case. However, in tropical and low-latitude regions, where wastewater temperatures can exceed 25 to 30 °C, short-sludge systems would normally nitrify; in fact,

it would be difficult to stop them doing so. For these situations, it is best to accept nitrification as inevitable and design the system accordingly. Furthermore, it would be advantageous to include a small primary anoxic zone (~15-20% anoxic mass fraction, see Chapter 5) in the system to denitrify a considerable proportion of the nitrate generated even if N removal

is not required; this increases the minimum sludge age for nitrification, reduces oxygen demand, recovers alkalinity and reduces the risk of sludge flotation and high effluent COD due to denitrification on the SST bottom.

Bio-P removal is possible at short sludge ages of 3 to 5 days; the phosphate-accumulating organisms (PAOs) are relatively fast-growing heterotrophs. In the absence of nitrification, an unaerated zone would be anaerobic (*i.e.* no nitrate or oxygen present or entering it) and provided the readily-biodegradable (RB) COD and short-chain fatty acids (SCFAs) are available from the influent, biological excess P removal will take place. The original Phoredox system developed by Barnard (1976) is based on such a two-reactor anaerobic-aerobic system. The minimum sludge age for EBPR is temperature-dependent, increasing as temperature decreases and is approximately 3 to 5 days at 14 to 20 °C (Mamais *et al.*, 1992). At these temperatures, the minimum sludge age for nitrification is significantly longer than that for EBPR, so that nitrification would generally not take place with the result that the adverse effect of nitrate on the EBPR would be absent. However, in warmer climates the minimum sludge ages for nitrification and EBPR are similar, and ensuring a low nitrate recycle to the anaerobic reactor by including also anoxic zones is essential if EBPR is required (Burke *et al.* 1986). If EBPR is not required, the nitrification changes the two-reactor unaerated-aerated system from a P-removal one to an N-removal one.

Currently there is renewed interest in the 1st or A stage of the 2-stage or AB process, where the A stage is a high-rate activated-sludge (HRAS) system for organics removal ($SRT < 1.5$ d) and the B stage a long sludge-age system for N removal (Haider *et al.*, 2006). This renewed interest has driven low energy wastewater treatment by minimizing energy consumption by aeration and maximizing organics capture as primary and/or secondary sludge for anaerobic digestion and energy recovery (Jimenez *et al.*, 2015; De Graaff *et al.*, 2016) and replacing the B stage by mainstream anammox for N removal (Zhang *et al.*, 2019).

4.13.1.2 Aerated lagoons

Aerated lagoons, different from aerated oxidation ponds where oxygenation is supplemented by algae, are essentially high-rate activated sludge systems because the oxygen demand is totally supplied by aerators. There are essentially two types of aerated lagoons, suspension mixed and facultative. Suspension-mixed aerated lagoons have sufficient energy input per unit volume by the aeration equipment to keep the sludge in suspension. In facultative lagoons this energy input is insufficient and settlement of solids onto the lagoon floor takes place. The biodegradable solids in the sludge layer so formed degrade anaerobically, as in an oxidation pond.

Kinetically, suspension-mixed lagoons are flow-through activated sludge systems, and can be modelled as such (Marais *et al.*, 2017). Their nominal hydraulic retention time equals their sludge age and the waste (Q_w) and effluent (Q_e) flows are one and the same, and equal to the influent flow (Q_i). Hence the volume of the aerated lagoon per unit COD load is very large compared with the conventional short sludge-age systems, which have hydraulic retention times approximately 1/20th of the sludge age.

The effluent from a suspension mixed aerated lagoon has the same constituents as the mixed liquor in the lagoon. The COD removed from the system via the oxygen demand is relatively small so that the COD in the effluent is generally unacceptable for discharge into receiving waters. In fact, the principal objective of all short-age plants is to act as biologically-assisted flocculators. This biologically transforms the influent soluble biodegradable organics to settleable organism mass, and enmeshes with this the influent biodegradable and unbiodegradable particulate organics to form a settleable sludge which allows effective liquid-solid separation. In conventional short-sludge-age plants, the waste sludge is transferred to the sludge treatment facility; in the aerated lagoon systems, the effluent (with the waste sludge) usually flows to a second pond, *i.e.* an oxidation pond or a facultative aerated lagoon, to

allow the now readily settleable particulate material to settle to the lagoon floor to produce a relatively solids-free and low COD effluent. The sludge that accumulates on the tank floor undergoes anaerobic stabilization. Aerated lagoons find application principally as low-technology industrial waste treatment systems where organic strengths are high, the load varies seasonally and nitrification is not required. However, treating these waste waters in different types of anaerobic digestion systems is becoming more important to benefit from their better effluent quality, water re-use, energy recovery and reduced greenhouse-gas emissions.

4.13.2 Intermediate sludge ages (10 to 15 days)

Where nitrification is obligatory because of a strict effluent FSA concentration standard, this will govern the minimum sludge age of the activated sludge system. For nitrification, the sludge ages required are 5 to 8 times longer than those for COD removal only, depending on the temperature. In temperate regions where water temperatures can fall below 14 °C, the sludge age is not likely to be less than 10 to 15 days, taking due consideration of some unaerated zones in the reactor for denitrification (and biological P removal). In this range of sludge age, the effluent COD concentration no longer plays a role in the design. For sludge ages longer than approximately 4 days, protozoan organism predation of free swimming bacteria is high and flocculation good so particle dispersion is low. Also, virtually all soluble biodegradable organics are broken down, with the result that the effluent COD (or BOD) concentration remains approximately constant at its lowest achievable value, *i.e.* the unbiodegradable soluble COD concentration. The effluent ammonia concentration also plays a minor role in design because the nitrification kinetics are such that once nitrification is achieved, it is virtually complete provided sufficient oxygen is supplied. Even though the effluent standards may require an effluent ammonia concentration, say <10 mgFSA-N/l, once nitrification takes place the concentration is not likely to be greater than 2 to 4 mgN/l. Consequently, for

nitrification the sludge age of the system is fixed principally by the requirement for nitrification. The method for calculating the minimum sludge age for nitrification is given in Chapter 5, Section 5.1.7. Once a sludge age of say 25% longer than the minimum is selected, the effluent FSA concentration is affected more by the system operating conditions than by the nitrification process itself, *i.e.* oxygen supply limitations, variation in ammonia load, uncontrolled loss of sludge, and pH of the mixed liquor. With low alkalinity wastewaters, nitrification can cause a significant reduction in effluent pH, often as low as 5. This not only causes problems with the nitrification process itself, *i.e.* non-compliance with the effluent ammonia standard, but also produces aggressive effluents which can do considerable damage to concrete surfaces. To reduce these problems and derive the other advantages of oxygen and alkalinity recovery (see below), the policy of deliberate biological denitrification is advocated whenever nitrification is likely, even if N removal is not required. However, once nitrification is required and biological denitrification is incorporated in the system, sludge ages longer than 10 to 15 days may be required and the system falls into the long-sludge-age category.

In nitrifying aerobic activated sludge plants, there is always the possibility of denitrification in the SST. This problem is exacerbated by the system control procedure of abstracting the waste sludge from the settling tank underflow (see Section 4.12.1). At low underflow recycle ratios, sludge retention in the SST is long which leads to denitrification (Figure 4.13). Henze *et al.* (1993) estimated that between 6-8 and 8-10 mgN/l nitrate needs to be denitrified to cause sludge flotation at 10 and 20°C, respectively. The concentration of nitrate denitrified increases as (i) sludge retention time in the SST increases, which is dependent on the recycle ratio and peak flow conditions, (ii) the active fraction of the sludge increases, *i.e.* is larger at shorter sludge ages (Figure 4.3B), (iii) temperature increases, and (iv) the mass of unutilized enmeshed biodegradable organics increases which is higher at shorter sludge ages and largest at the peak load condition (Ekama *et al.*, 1997).

The above demonstrates that for plants where nitrification takes place, the SST should not serve the dual purpose of solid-liquid separation and thickening of waste sludge, that hydraulic control of sludge age should be employed and that deliberate denitrification should be included in the system (see Section 4.12). These modifications will ameliorate the problem of sludge flotation by denitrification in the SST, but may not completely eliminate the root cause, *i.e.* high nitrate concentrations in the mixed liquor.

In order to reduce the construction cost of the activated sludge system, reductions in sludge age need to be made. Moreover, a reduction in sludge age also increases both biological N and P removal per mass organic load (WRC, 1984; Wentzel *et al.*, 1990) and this would be particularly beneficial for low-temperature wastewaters (10-15 °C) where nitrification is required. Reduction in SRT has been achieved in various ways such as adding nitrifier biofilm carriers to the aerobic reactor or selectively wasting the light fraction of the activated sludge with hydro-cyclones which enriches the activated sludge with the heavier fraction containing aerobic ammonia oxidizing bacteria (AOB), PAO and anaerobic ammonia oxidizing bacteria (AMX) (Ekama, 2015).

To try to reduce the sludge age required for nitrification, and hence the biological reactor volume per Ml of wastewater treated, internal fixed media have been placed in the aerobic reactor (Wanner *et al.*, 1988; Sen *et al.*, 1994; Ødegaard *et al.*, 2014). The nitrifiers that grow on these fixed media are not subject to the mixed liquor sludge age and aerobic mass fraction with the result that both can be reduced. However, the effectiveness of the internal fixed media has not been as good as expected, and they yield a rather low benefit/cost ratio.

Successful reduction of sludge age down to 8 to 10 days has been achieved with external nitrification (Bortone *et al.*, 1996; Sorm *et al.*, 1997; Hu *et al.*, 2000) and this system is starting to find application at full-scale (Vestner and Günthert, 2001; Muller *et al.*, 2006). With external nitrification, the nitrification process is removed completely from the suspended

activated sludge and transferred to an external fixed medium system such as a trickling filter. With nitrification independent of the BNR activated-sludge mixed liquor, the sludge age can be reduced to around 8 to 10 d. Such a reduction reduces the biological reactor volume requirement per Ml of wastewater treated by approximately a 1/3rd without negatively impacting either biological N or P removal. Moreover, the sludge settleability improves significantly (DSVI~60-80 ml/g) compared with conventional BNR systems, which further increases the capacity of the system (Hu *et al.*, 2000).

Comparing intermediate sludge age plants with high rate plants, the oxygen demand per kg COD (including nitrification) is doubled (except with external nitrification, for which it is halved), the system volume is 3 to 4 times larger, the daily sludge mass wasted is reduced by 40%, and the active fraction is much lower. Intermediate-sludge-age plants are much more stable than high-rate plants, requiring less sophisticated control techniques or operator intervention (excepting external nitrification) thereby making these plants more suitable for general application.

At intermediate sludge ages, the active fraction of the waste sludge is still too high for direct discharge to drying beds. Consequently some form of waste sludge stabilization would need to be incorporated in the wastewater treatment plant, *e.g.* aerobic or anaerobic digestion. The former has the advantage of ease of operation and if operated at high MLSS concentration (> 2%) and with intermittent aeration, produces low N and P concentrations in the dewatering liquor (Mebrahtu *et al.*, 2008), but has the disadvantage of high energy costs for oxygen supply. The latter has the advantage of energy generation from the biogas but the disadvantage of complexity of operation and high N and P concentrations in the dewatering liquor. Even with energy recovery by anaerobic digestion of waste sludge, because of the low mass of sludge wasted from the activated sludge plant and high oxygen demand per kg COD load, energy self-sufficiency at intermediate sludge ages is not possible. However, in large plants (approximately

500,000 PE) where technical supervision and operator expertise are of a high level, energy costs can be reduced by gas production from anaerobic digesters and can probably be justified economically, particularly if energy costs continue to increase as they have over the past decade. Ekama (2009) found that the carbon dioxide emission originating from the influent organics from two widely-differing treatment plants treating the same wastewater is virtually the same if the residual biodegradable organics (COD) in the final disposed sludge is the same, viz (i) a long sludge-age (30 d, Table 4.6) extended-aeration activated sludge system treating raw wastewater and (ii) a short sludge-age (8 d, Table 4.6) activated sludge system treating settled wastewater with anaerobic digestion of primary sludge and aerobic or anaerobic digestion of wastewater activated sludge with beneficial combustion/flaring of methane gas.

4.13.3 Long sludge ages (20 days or more)

4.13.3.1 Aerobic plants

Long-sludge-age aerobic plants are usually called extended aeration plants. The principal objective of long sludge systems is to obviate primary (1^{st}) and secondary (2^{nd}) sludge treatment. These plants therefore treat raw wastewater and the sludge age is chosen so that the active fraction (or residual biodegradable organics) of the waste sludge is sufficiently low to allow its direct discharge to sludge drying beds. The sludge age required to produce sludge sufficiently stable so as not to generate odour problems is uncertain and will depend on the temperature and climatic conditions, i.e. whether or not the sludge can be dried sufficiently quickly before it starts smelling, probably exceeding 30 days.

Interestingly, from a survey of the residual biodegradable organics in wastewater sludges treated by different sludge stabilization systems, Samson and Ekama (2000) found that aerobically-digested waste activated sludge contained the lowest residual biodegradable organics (10%) compared with wet-air oxidized (Zimpro) and anaerobically-digested primary sludges (25-60 %, Figure 4.17).

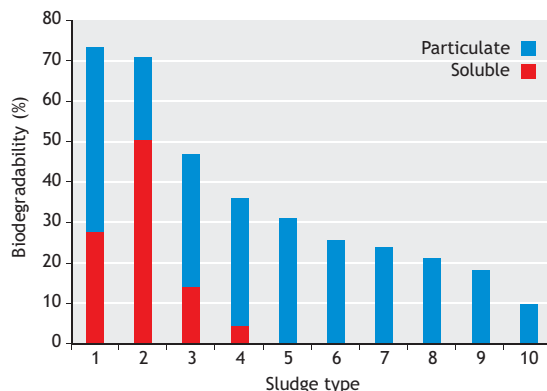


Figure 4.17 Percentage of residual biodegradable organics remaining in stabilised wastewater sludges treated with different stabilization system types. X-axis markers: (1) raw unsettled wastewater, (2) Zimpro humus + 10 - high soluble COD, (3) anaerobically-digested 10 + WAS - high VFA, (4) anaerobically-digested 10 only - high VFA, (5) anaerobically-digested 10, 1st stage - low VFA, (6) Zimpro humus + 10 - low soluble COD, (7) anaerobically-digested 10, 2nd stage - low VFA, (8) DAF thickened WAS, (9) anaerobically-digested 10 + WAS, single stage - low VFA and, (10) aerobically-digested WAS.

4.13.3.2 Anoxic-aerobic plants

Once the sludge age exceeds 20 to 25 days, nitrification is inevitable and it is advisable for the reasons cited above to incorporate denitrification in the system, which at these long sludge ages would not affect the stability of nitrification. Furthermore, if required, EBPR can also be included for little extra cost. In fact, biological N and P removal rates are significantly higher with raw wastewater than the settled wastewater due to the higher organic load. To include N (and P) removal, the reactor is subdivided into unaerated (anoxic and anaerobic) and aerated zones in a variety of configurations. Denitrification takes place in the unaerated but mixed zones receiving nitrified mixed liquor via recycles from the aerated zones to give what is referred to as nitrification denitrification (ND) systems. The ND systems include 4 stage Bardenpho, which incorporates primary and secondary anoxic reactors, Modified Ludzack Ettinger (MLE), which incorporates only a primary anoxic reactor, Orbal®, Carousel® and oxidation ditch systems in which the anoxic zones created are along

different lengths of the same long channel reactor, or in intermittently decanted extended aeration (IDEA) systems. While incorporation of denitrification imposes some additional constraints on the design at long sludge age, these are minor provided the aeration capacity of the plant is sufficient to ensure efficient nitrification under all expected conditions (see Chapter 5).

4.13.3.3 Anaerobic-anoxic-aerobic plants

When the EBPR is required, an initial anaerobic reactor is included in the configuration that receives the influent wastewater but minimal oxygen and nitrate via the sludge recycles. For EBPR, assurance of a zero-nitrate discharge to the anaerobic zone is critical for achieving good P removal and is an additional constraint on the design when including EBPR in extended-aeration systems. The extent of EBPR achieved will depend on a number of factors, mainly the influent readily-biodegradable (RB) COD concentration, the TP/COD ratio and the degree to which nitrate can be excluded from the anaerobic reactor, which depends on the influent TKN/COD ratio.

The waste sludge from extended aeration systems including EBPR has the potential to release high P concentrations. This can be dealt with in specially designed dewatering/drying beds with sand filters under drains and weir overflows, which allow the drying bed also to operate as a dewatering system. While discharging waste sludge directly to the drying bed, the under drain and overflow are monitored for P concentration and when this gets to say 5 mgP/l, sludge wastage to the drying bed and the return of supernatant to the head of the works is stopped. The relatively small volume of high P liquor that drains from the drying bed thereafter is either chemically treated or irrigated at the plant site. The dewatering capability of the drying bed allows significantly more sludge to be discharged to it than drying beds without these dewatering features.

4.13.4 Dominant drivers for activated sludge system size

In the above section, some of the considerations for selection of the activated sludge system sludge age were set out because this is the most fundamental and important decision in its design. Sludge age is the main driver that governs effluent quality and size of the activated sludge system. Generally, the higher the effluent (and waste sludge) quality required from the system, the longer the sludge age, the larger the biological reactor and the more wastewater characteristics that need to be known (Figure 4.18).

For organic-matter removal only, the sludge age of the system is short and hence the reactor volume small. Essentially only the organic (COD) load and unbiodegradable particulate and soluble COD fractions need to be known. The organic load and unbiodegradable particulate COD concentration heavily affect sludge mass in the reactor and daily sludge production and the unbiodegradable soluble COD concentration fixes the filtered effluent COD concentration from the system. Also the organic load fixes the daily oxygen demand and the peak hydraulic load fixes the secondary settling tank surface area. If nitrification is required from the system, more wastewater characteristics are required to be known. The most important of these are the maximum specific growth rate of the nitrifiers at the standard wastewater temperature of 20 °C ($\mu_{\text{ANO},20}$) and the minimum wastewater temperature (T_{min}), both of which fix the minimum sludge age for nitrification ($\text{SRT}_{\text{min,NIT}}$). The system sludge age (SRT) must be selected longer than the minimum for nitrification and the higher the system - to - minimum - sludge - age ratio ($\text{SRT}/\text{SRT}_{\text{min,NIT}}$), the lower the effluent ammonia concentration and the more damped its variation in response to nitrogen-load variation. Also required for nitrifying systems is the daily nitrogen load (both TKN and FSA) so that the components making up the N material in the influent can be determined. Note that for nitrification the maximum specific growth rate of the nitrifiers is regarded as a wastewater characteristic and not a model kinetic constant because it is different in different wastewaters.

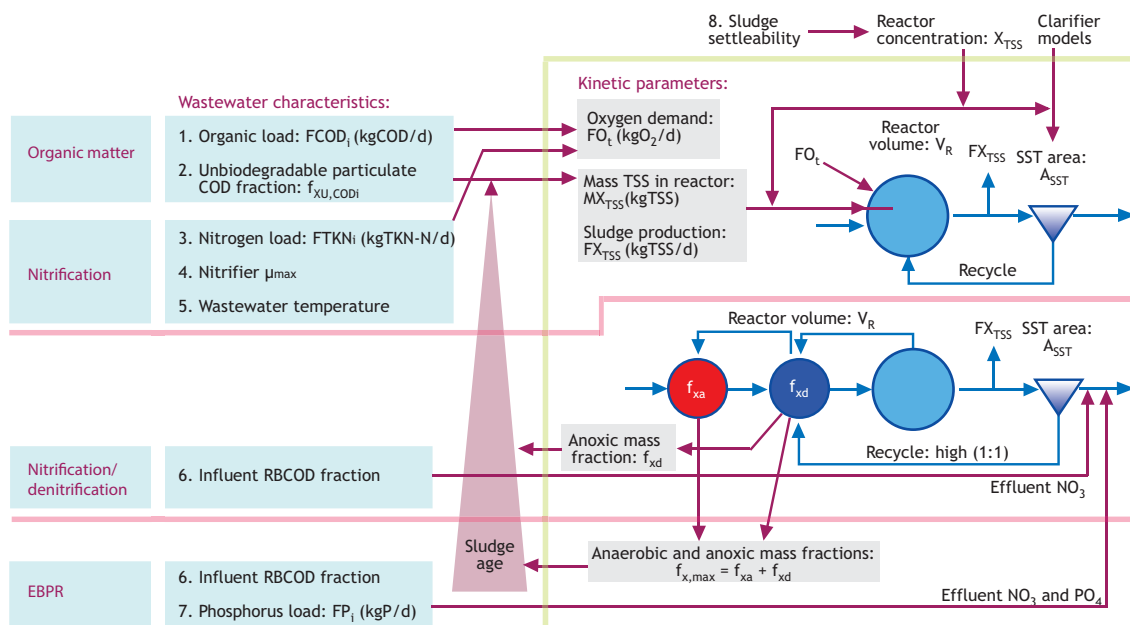


Figure 4.18 Important wastewater characteristics required to be known for different activated sludge systems: fully aerobic, nitrification, nitrification-denitrification and EBPR, and also the inter-relationships that affect sludge age and effluent quality.

With biological nitrogen removal (nitrification and denitrification, ND), a part or the biological reactor (the anoxic mass fraction, f_{xd}) is intentionally not aerated. The larger the anoxic mass fraction, the more nitrate can be denitrified but the longer the minimum sludge age for nitrification becomes over that for fully aerobic conditions. Thus for ND systems, the biological reactor becomes larger because the required sludge ages become longer. Also an additional wastewater characteristic needs to be known, *i.e.* the influent RBCOD concentration because a high proportion (up to half) of the nitrate denitrified in the primary anoxic reactor is due to this wastewater constituent: if the influent RBCOD concentration is not known, the effluent nitrate concentration cannot be calculated accurately.

With EBPR, the daily wastewater phosphorus load (both total P and ortho-P) needs to be known so that the components making up the P material in the influent can be determined. With EBPR the influent

RBCOD concentration is very important and establishes the extent of biological P removal that can be achieved. If the influent RBCOD concentration is not known, the biological P removal that can be achieved cannot be calculated accurately. The influent RBCOD is indirectly the food source for the phosphate-accumulating organisms (PAOs) that mediate the EBPR process. The purpose of the anaerobic zone, which receives the influent wastewater, is to allow the PAOs to take up the volatile fatty acid (VFA) fermentation products generated from the influent RBCOD. Nitrate (or dissolved oxygen, DO) which enters the anaerobic zone results in utilization of some of the influent RBCOD by ordinary heterotrophic organisms (OHOs), which reduces the VFA products available to the PAOs and hence the biological P removal. The difference between the influent P concentration and the EBPR that can be achieved establishes the effluent P concentration.

Very low nitrate (and DO) concentrations in the recycles entering the anaerobic zone are essential for maximum EBPR. This imposes important requirements on the denitrification required in the anoxic zones. If the influent TKN/COD concentration ratio is too high, then low nitrate concentrations cannot be achieved in the anoxic zone(s) and therefore methanol dosing may be required. High N removals in the anoxic zones requires large anoxic reactor(s), which together with the anaerobic zone, results in large unaerated mass fractions, which in turns requires long sludge ages to ensure nitrification. Unless specific strategies are applied to keep the sludge age low, such as external nitrification or adding fixed media into the aerobic zone to reduce the system sensitivity to the minimum sludge age for nitrification, NDEBPR systems will have long sludge ages, especially where wastewater minimum temperatures are low.

The above overview demonstrates that wastewater characteristic determination is the most important aspect of modelling wastewater treatment plants, whether using steady-state or dynamic-simulation models. Uncertainty in wastewater characteristics (and sludge settleability) results in a commensurate uncertainty in oxygen demand, sludge production, reactor volume and effluent quality. So uncertainty/sensitivity analyses should be applied to the wastewater characteristics rather than to the kinetic and stoichiometric parameters of the model(s). In fact, only rarely should the kinetic and stoichiometric parameters of the model be changed (except the maximum specific growth rate of nitrifiers which is regarded as a wastewater characteristic.) Fitting all the effluent quality concentrations, sludge production and oxygen demand to laboratory, pilot and full-scale plant data can be achieved by changing the wastewater characteristics only, provided the data conform to mass balances (water, COD, N and P). However, more often than not, model predictions cannot be made to conform to measured data because the measured data do not conform to mass balance and continuity principles. Only when the data conform to

mass balance and continuity principles and changing the wastewater characteristics cannot yield a close correlation between the model predictions and the measured data, should kinetic and stoichiometric parameters of the model be changed, but such change(s) should be based on bioprocess basics and not simply because 'it makes the model fit'.

4.13.5 Some general comments

In BNR systems of any sludge age, aeration control is a particularly vexing problem under cyclic flow and load conditions, because the system is affected by either too high or too low DO concentrations in the aerobic zone. Too high DO concentrations are unnecessarily expensive and result in oxygen recycle to the anoxic (and the anaerobic zone if EBPR is included), thereby reducing the potential for N and P removal; too low DO concentrations cause nitrification efficiency to decline and possibly poor settling sludges to develop.

While some effective DO control systems have been developed over the years, the cost of providing aeration capacity and SST surface area for the peak flow has prompted research into alternative control solutions such as flow and load equalization. Furthermore, most of the diurnal variation in system variables such as ammonia, nitrate and phosphate concentration is not induced by the biological processes but by the hydraulic flow variation. To minimize hydraulic flow variation, an equalization tank is provided upstream of the activated sludge system and outflow from this tank is controlled in such a manner that the cyclic fluctuations in flow and load are damped to very small values. The tank is controlled by a microcomputer which calculates the tank outflow rate which best damps the projected inflow of the next 24 hours. This flow-equalization approach has been tested at the Goudkoppies BNR plant (Johannesburg, RSA) and showed great potential for reducing aeration and other control problems in nutrient removal plants (Dold *et al.*, 1982, 1984).

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NOMENCLATURE

Symbol	Description	Unit
a	Mixed liquor recycle ratio (Q_a/Q_i)	-
A_{SST}	Surface area of the secondary settling tank	m^2
b_{OHO}	Specific rate of endogenous mass loss of OHOs	d^{-1}
C_{br}	Bioreactor cost constant	-
$COD_{b,i}$	Influent biodegradable COD	$mgCOD/l$
COD_i	Influent total COD	$mgCOD/l$
COD_e	Effluent total COD	$mgCOD/l$
$COD_{e(filt)}$	Effluent soluble COD	$mgCOD/l$
$COD_{e(unfilt)}$	Effluent total COD	$mgCOD/l$
C_{sst}	Secondary settling tank cost constant	-
$DSVI$	Diluted sludge volume index	$ml/gTSS$
f_a	Fraction of OHOs in the activated sludge	$mgVSS/mgVSS$
$f_{at,OHO}$	Fraction of OHOs in the sludge as TSS	$mgVSS/mgTSS$
$f_{av,OHO}$	Fraction of OHOs in the sludge	$mgVSS/mgVSS$
$f_{cv,OHO}$	COD to VSS ratio of the sludge	$mgCOD/mgVSS$
$f_{fss,OHO}$	Inorganic content of OHOs	$mgISS/mgCOD$
f_q	Peak flow factor (PWWF/ADWF)	l/l
$f_{SU,CODi}$	Soluble unbiodegradable fraction of total influent COD	-
f_{VT}	Ratio of VSS over TSS of the sludge	$mgVSS/mgTSS$
$f_{XU,CODi}$	Particulate unbiodegradable fraction of total influent COD	-
$f_{XE,OHO}$	Unbiodegradable fraction of the OHOs	$mgCOD/mgCOD$
$FCOD_{b,i}$	Daily flux of influent biodegradable COD	$mgCOD/d$
$FCOD_e$	Flux of effluent COD	$mgCOD/d$
$FCOD_i$	Daily flux of influent total COD	$mgCOD/d$
$FCOD_{VSS}$	Daily flux of particulate organic matter produced	$mgCOD/d$
FO_e	Daily flux of oxygen utilised	mgO_2/d
$FX_{FSS,i}$	Daily flux of influent particulate inorganic matter	$mgISS/d$
FX_{TSS}	Daily flux of total solids produced	$mgTSS/d$
$FX_{U,i}$	Daily flux of influent particulate unbiodegradable COD	$mgCOD/d$
$FX_{Uv,i}$	Daily flux of influent particulate unbiodegradable matter	$mgVSS/d$
FX_{VSS}	Daily flux of volatile solids produced	$mgVSS/d$
ISS	Inorganic suspended solids matter of activated sludge	$mgISS/l$
HRT_a	Actual hydraulic retention time	d

HRT_n	Nominal hydraulic retention time	d
MX_{OHOv}	Mass of OHO in the bioreactor	mgVSS
$MX_{E,OHOv}$	Mass of endogenous residue in the bioreactor	mgVSS
MX_{FSS}	Mass of influent particulate inorganic matter in the bioreactor	mgISS
MX_{TSS}	Mass of solids in the bioreactor	mgTSS
MX_{Uv}	Mass of influent unbiodegradable matter in the bioreactor	mgVSS
MX_{VSS}	Mass of volatile suspended solids in the bioreactor	mgVSS
O_c	Carbonaceous oxygen utilisation rate	mgO ₂ /l.d
P_{br}	Bioreactor power cost constant	-
P_{sst}	Secondary settling tank power cost constant	-
Q_a	Mixed liquor recycle flowrate	l/d
Q_e	Effluent flowrate	l/d
Q_i	Influent flowrate	l/d
$Q_{i,ADWF}$	Influent flowrate (for average dry weather)	l/d
$q_{i,PWWF}$	SST overflow rate at peak wet weather flow	m/h
$Q_{i,PWWF}$	SST flowrate at peak wet weather flow	m ³ /h
Q_s	Sludge recycle flowrate	l/d
Q_w	Wastage flowrate from bioreactor	l/d
τ_{hin}	Sludge settling constant	l/g
SRT	Sludge retention time	d
s	Sludge underflow recycle ratio (Q_s/Q_i)	-
S_S	Soluble (readily) biodegradable (RB) COD	mgCOD/l
$S_{U,e}$	Effluent (unbiodegradable) soluble COD	mgCOD/l
$S_{U,i}$	Influent soluble unbiodegradable COD	mgCOD/l
$SSVI_{3.5}$	Stirred specific sludge volume index at 3.5 g TSS/l	mL/gTSS
V_0	Initial settling velocity	m/h
V_R	Volume of bioreactor	l
V_s	Zone settling velocity	m/h
X	Particulate matter of activated sludge	mgTSS/l
X_{OHOv}	OHO biomass	mgVSS/l
$X_{E,OHOv}$	Endogenous residue from OHOs in activated sludge	mgVSS/l
$X_{FSS,i}$	Influent inorganics concentration	mgISS/l
X_{sR}	Suspended solids concentration in the sludge recycle from the SST	mgTSS/l
X_S	Slowly biodegradable (SB) particulate influent COD	mgCOD/l
$X_{S,i}$	Influent particulate unbiodegradable COD	mgCOD/l
X_{TSS}	Particulate matter of activated sludge	mgTSS/l
X_U	Influent unbiodegradable matter in activated sludge	mgCOD/l
X_{Uv}	Influent unbiodegradable matter in activated sludge	mgVSS/l
X_{VSS}	Organic matter of activated sludge	mgVSS/l
$X_{VSS,e}$	Effluent particulate volatile matter	mgVSS/l
Y_{OHO}	COD yield of OHOs	mgCOD/mgCOD
Y_{OHOv}	VSS yield of OHOs	mgVSS/mgCOD

Abbreviation	Description
ADWF	Average dry weather flow
AS	Activated sludge
BOD	Biological oxygen demand
BNR	Biological nutrient removal
COD	Chemical oxygen demand
DSVI	Diluted sludge volume index
DO	Dissolved oxygen
EBPR	Enhanced biological phosphorus removal
ESS	Effluent suspended solids
F/M	Food to microorganisms ratio
HRT	Hydraulic retention time
FSA	Free and saline ammonia
IDEA	Intermittently decanted extended aeration
ISS	Inorganic component of the settleable solids mass
LF	Load factor
MLSS	Mixed liquor suspended solids
MLVSS	Mixed liquor volatile suspended solids
OHOs	Ordinary heterotrophic organisms
ND	Nitrification-denitrification
PAOs	Phosphorus accumulating organisms
PE	Person equivalent
PST	Primary settling tank
PWWF	Peak wet weather flow
RBCOD	Readily biodegradable COD
SRT	Sludge retention time (sludge age)
SS	Suspended solids
SST	Secondary settling tank
SVI	Sludge volume index
SV	Settled volume
SSVI	Stirred sludge volume index
TKN	Total Kjeldahl nitrogen
TSS	Total suspended solids
VFAs	Volatile fatty acids
VSS	Volatile suspended solids
WAS	Waste activated sludge

Greek symbols	Explanation	Unit
$\theta_{b,OHO}$	Arrhenius temperature coefficient for the endogenous respiration rate of OHOs	-
ϕ	Secondary settling tank diameter	m

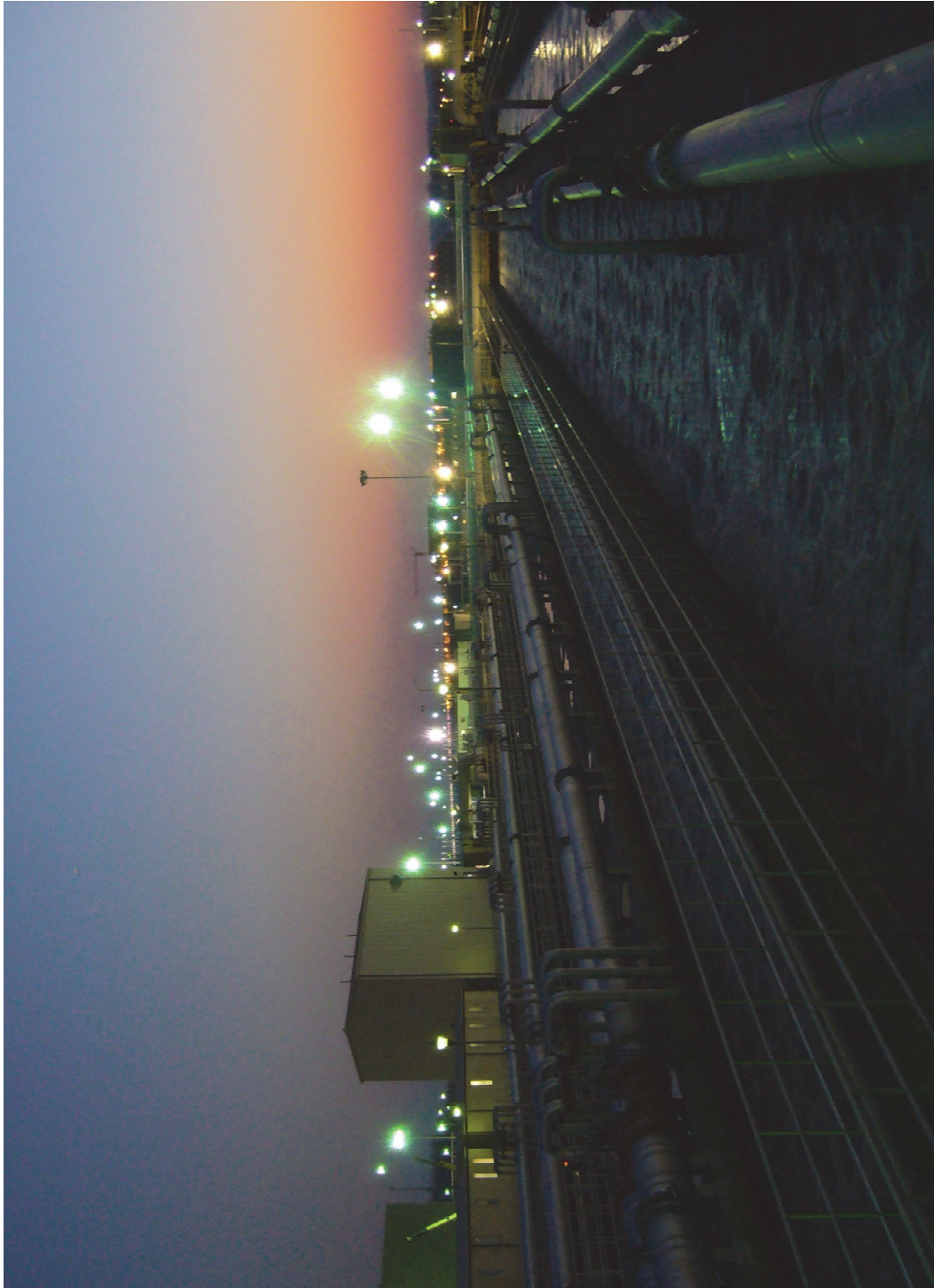


Figure 4-19 'The night shift' - microorganisms 'at work', WWTP Blue Plains, Washington DC (Photo: D. Rosso)