New German design guideline for single stage activated sludge plants

R. Kayser
Adolf-Bingel-Str. 2, D 38116 Braunschweig, Germany

Abstract The German design guideline A 131 "Design of single stage activated sludge plants" was amended in 1999. The main changes of the guideline from 1991 are outlined. The design procedure for plants with nitrogen and phosphorus removal is presented.
Keywords Activated sludge process; nitrogen removal; phosphorus removal; wastewater treatment

Introduction

When in 1988 the German Government planned to require nitrogen and phosphorus removal for all larger wastewater treatment plants, the ATV committees 2.5 "Settling tank design" and 2.6 "Aerobic biological wastewater treatment processes" (of which the author presently is the chairman) started to work out the design guideline A 131 "Design of single stage activated sludge plants for 5,000 p.e. and more", which was issued in February 1991 (ATV, 1991).

Since at that time there were only a few full scale plants for nitrogen and phosphorus removal, the guideline was mainly based on pilot plant results and research findings. In the meantime a larger number of full scale plants is in operation. For an amendment of the guideline, therefore, a broader data base was available. After one year of work of the two committees the "yellow" print of the amended guideline A 131 was issued in April 1999 (ATV, 1999).

The main changes are:

- The estimation of design flows and loads was taken out. A separate guideline on the estimation of design flows and loads for sewerage systems and wastewater treatment plants will be issued by ATV.
- Integration of design for enhanced biological phosphorus removal as well as the design of aerobic selectors.
- Improvement of the denitrification capacity and new coefficients for the calculation of the oxygen requirements.
- Option to base the calculation of the sludge production and the oxygen requirement for carbon removal on COD.
- Improvements in final settling tank design.
- A disk with the design calculations will be added to the guideline.

Design procedure

A main step in the design is the assumption of the sludge volume index (SVI) because it influences the size of the final settling tank as well as the biological reactor. Since it depends not only on the wastewater characteristics but also on the process configuration and the sludge age (SRT) the selection of the process has to precede. The next step then is the design of the final settling tank because thereby the return activated sludge (RAS) flow rate as well as the solids concentration of RAS and finally the mixed liquor suspended solids concentration (MLSS) are determined. The dimensioning of the biological reactor and the estimation of excess sludge production as well as the oxygen requirements follows. A last step may be

Water Science and Technology **Vol 41 No 9 pp 139–145**

the variation of MLSS in order to optimize the sizes of the biological reactor and the final settling tank, respectively.

Process options

The processes for **nitrogen removal** mainly applied in Germany are (Figure 1):

1. Pre-anoxic zone denitrification, in German "Vorgeschaltete Denitrifikation" (Ludzack and Ettinger, 1962). By internal recirculation and RAS nitrate is transferred into the anoxic zone. It is important to avoid high DO at the nitrification tank effluent. The denitrification zone as well as the nitrification zone may be subdivided. The removal rate of nitrate can be limited by the flow rate of the internal recirculation.
2. Step-feed process, in German "Kaskadendenitrifikation" (Miyaji *et al.*, 1980). It can be visualized as a series of pre-anoxic zone systems, whereby RAS is introduced into the first anoxic zone. MLSS is highest in the first zone and lowest in the last. The wastewater flow usually is not equally distributed but stepwise decreasing in order to maintain similar sludge loading rates in all zones. The flow fraction diverted to the last zone ($x \cdot Q$) in conjunction with the RAS flow rate may limit the nitrate effluent concentration. Internal recirculation usually is not applied.
3. Simultaneous nitrification denitrification process (e.g. Matsché, 1977), usually performed in circulating flow closed loop tanks with horizontal shaft surface aerators (mammoth rotors) or with vertical shaft surface aerators (carousel system). The aerators have to be automatically controlled (e. g. by on-line nitrate monitoring) in order to establish the appropriate fractions of volume for nitrification and denitrification. Besides the aerators mixers may be installed in order to avoid sludge settling at low load periods.

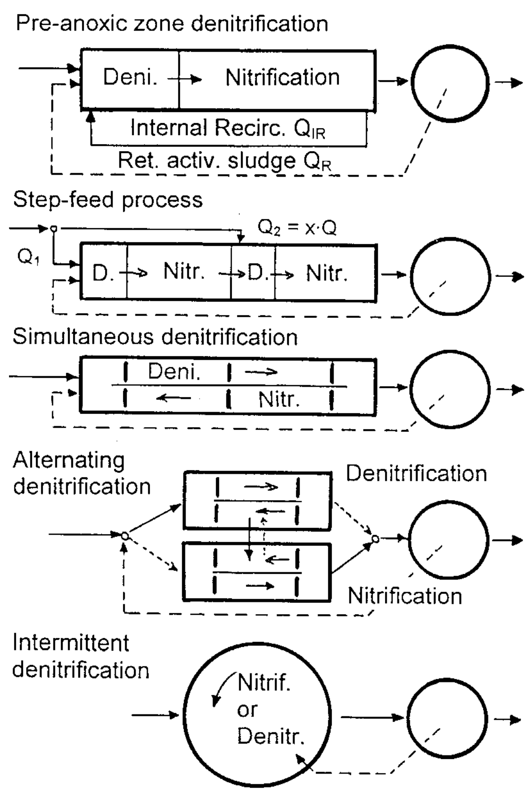

Figure 1 Processes for nitrogen removal

4. Alternating nitrification denitrification process (BioDenitro process, Tholander, 1977). A pair of interconnected tanks is fed alternating with wastewater and RAS. Most of the feeding period the tank is mixed for denitrification. The other tank from which the mixed liquor is diverted to the final settling tank is aerated for nitrification. Each tank, therefore, is equipped with an aeration and a mixing installation. The valves or weirs for directing the flow may be timer operated.
5. Intermittent nitrification denitrification process. A biological reactor equipped with an aeration installation and a mixing device is continuously fed with wastewater and RAS. Aeration is periodically switched on and off for nitrification and denitrification, respectively.

Each process may be operated after **primary sedimentation** which is usual, but also without primary sedimentation which anyway is the case if aerobic co-stabilization of the sludge (extended aeration) is anticipated.

For **phosphorus removal** each of the nitrogen removal processes may be preceded by an anaerobic mixing tank for enhanced biological phosphate removal. Since it is assumed that in the mixing tank designed for a recommended contact time of 0.5 to 0.75 hours (for wastewater and RAS) about 0.02 to 0.025 mg P per mg BOD_5 resp. 0.01 to 0.0125 mg P per mg COD may be removed, in addition simultaneous phosphate precipitation generally is applied in order to fulfil the discharge requirements. Anaerobic mixing tanks with respect to filamentous growth do have effects like aerobic selectors.

If the wastewater is characterized by a high concentration of readily biodegradable COD and a high COD/N-ratio it is advisable to implement an **aerobic selector** in order to suppress filamentous growth. The selector volume may be calculated for a volumetric BOD_5 load of 10 kg $BOD_5/(m^3\ d)$, respectively 20 kg $COD/(m^3\ d)$.

Unfortunately neither anaerobic tanks nor aerobic selectors are capable to prevent the growth of nasty filaments like e.g. *Microthrix Parvicella*.

Secondary settling tank design calculations

After the biological treatment process is selected, the first step in secondary settling tank design is the assumption of the sludge volume index (Table 1).

The lower values may be chosen if:

- primary sedimentation is not implemented,
- an aerobic selector or an anaerobic mixing tank precedes the main biological reactor,
- the main biological reactor is subdivided (closer to plug flow conditions).

The next steps are:

- Assume the sludge thickening time ($t_{th} \leq 2.5$ h) of the bottom sludge and estimate the bottom sludge concentration X_{bot} (kg/m^3) depending on t_{th} and SVI:

$$X_{bot} = \frac{1000}{SVI} \cdot \sqrt[3]{t_{th}} \tag{1}$$

- Assume the return activated sludge concentration $X_R \sim 0.7 \cdot X_{bot}$ for scrapers and $X_R \sim 0.5$ to $0.7 \cdot X_{bot}$ for suction collectors.

Table 1 Approximate values for the sludge volume index

	Industrial and trade wastewater influence	
	Favorable	Unfavorable
With or without nitrification and denitrification	SVI=100–150	SVI=120–180
Extended aeration (co-stabilization of sludge)	SVI=75–120	SVI=100–150

- Assume the return activated sludge flow ratio $Q_R/Q \leq 0.7$ to 1.0 and calculate the sludge concentration of the settling tank inlet $X_F=(X_R \cdot Q_R)/(Q+Q_R)$.
  The settling tank surface area A_{ST} (m^2) is obtained as:

$$A_{ST} = \frac{Q}{q_A} = \frac{Q \cdot X_F \cdot SVI}{q_{SF}} \tag{2}$$

The sludge volume loading rate for horizontal flow settling tanks may be selected as $q_{SF} \leq 500$ l/(m^2 h) but the overflow rate q_A shall not exceed 1.6 m/h. The flow rate Q(m^3/h) is the design (maximum) flow rate for storm weather periods.

The depth h_{tot} (m) of horizontal flow secondary settling tanks is calculated as follows:

$$h_{tot} = 0.5 + q_A \cdot (1 + Q_R / Q) \cdot \left(\frac{0.5}{1 - X_F \cdot SVI / 1000} + \frac{0.45 \cdot X_F \cdot SVI}{500} + \frac{X_F \cdot t_{th}}{X_{bot}} \right) \tag{3}$$

The depth shall at least be 3.00 m. The side wall water depth of circular settling tanks shall at least be 2.50 m.

From Eqs. (2) and (3) it is obvious that the tank surface as well as the tank depth increase with increasing sludge settled volume $SSV_{30}=X_F \cdot SVI$ (ml/l). Since a high mixed liquor suspended solids concentration is favorable, for economic reasons the sludge volume index is the key parameter.

The guideline furthermore contains design calculations for the sludge collection systems as well as calculations for vertical flow settling tanks.

Biological reactor design calculations

For nitrogen removal plants the first step is to estimate the mean nitrate concentration to be denitrified $S_{NO3,D}$ (mg N/l) for dry weather conditions:

$$S_{NO3,D} = C_{N,0} - C_{orgN,E} - S_{NH4,E} - S_{NO3,E} - X_{orgN,WAS} \tag{4}$$

The following values may be assumed: Organic nitrogen effluent $C_{orgN,E}=2$ mg/l, ammonia nitrogen effluent $S_{NH4,E}=0$ to 2 mg/l and nitrogen used for build-up of biomass $X_{orgN,WAS}=0.04$ to $0.05 \cdot C_{BOD5,0}$, respectively 0.02 to $0.025 \cdot C_{COD,0}$. The effluent nitrate concentration has to be assumed considering the discharge requirements as well as the mode of inspection, e. g. grab sample, 2-h-composite or 24-h composite sample. For the German discharge requirements ($S_{inorgN,E,req}=18$ mg/l in grab sample) it is recommended to assume as daily mean $S_{NO3,E}=0.6$ to $0.8\ S_{inorgN,E,req}$. Since the influent BOD_5 is known the required denitrification capacity $S_{NO3,D}/C_{BOD,0}$ (mg N/mg BOD_5) can be calculated.

Table 2 Recommended values for $S_{NO3,D}/C_{BOD,0}$ for different anoxic volume fractions V_D/V at T=10 to 12°C

	$S_{NO3,D}/C_{BOD,0}$	
V_D/V	Pre-anoxic zone denitrification and equivalent processes*)	Simultaneous and intermittent denitrification*)
0.2	0.11	0.06
0.3	0.13	0.09
0.4	0.14	0.12
0.5	0.15	0.15

*Nitrate entering a preceding anaerobic mixing tank for enhanced biological phosphate removal will be completely removed independently from the temperature

The guideline contains equations to calculate the denitrification capacity as a function of the oxygen uptake rate and the anoxic volume fraction V_D/V, from which, in conjunction with experimental and full-scale results, the values in Table 2 were derived.

It is not recommended to design the plants for $V_D/V>0.5$. If due to the wastewater composition $S_{NO3,D}/C_{BOD,0}>0.15$ it is recommended to plan external carbon addition. The construction, however, should be delayed until full scale results show the necessity.

The sludge age $t_{S,T}$ (d) is calculated with V_D/V taken from Table 2:

$$t_{S,T} = SF \cdot 3.4 \cdot 1.103^{(15-T)} \cdot \frac{1}{1 - V_D / V} \tag{5}$$

A sludge age of 3.4 d at 15ºC (or 5.55 d at 10ºC) ensures that sufficient nitrifiers are maintained in the biological reactor. At a constant nitrogen loading rate and without any inhibitory influence almost complete nitrification should be achieved. The safety factor SF considers mainly the daily nitrogen load fluctuations but also to some extent fluctuations of the maximum growth rate and short range temperature and pH fluctuations, respectively. It is recommended to use SF=1.8 for smaller plants ≤20.000 p.e. and 1.45 for larger plants ≥100.000 p.e. For plant sizes in between SF has to be interpolated.

The temperature T (ºC) in Eq. (5) may be chosen in accordance with the requirements for nitrogen removal. In Germany e. g. nitrogen removal is required at T≥12ºC. If the reactor temperature at any time is higher than 12ºC the higher two week average of T may be used to calculate $t_{S,T}$. Usually, however, T=12ºC will be applied to obtain $t_{S,12}$. If in winter times the reactor temperature drops to e. g. 9ºC it shall then be shown that by reducing V_D/V nitrifiers will not be washed out.

$$V_D / V = 1 - \frac{SF \cdot 3.4 \cdot 1.103^{(15-T)}}{t_{S,12}} \tag{6}$$

If for very low temperatures or at plants with a small anoxic volume fraction VD/V<0 is obtained, it shall be set $V_D/V=0$. The safety factor then may be reduced to SF=1.2. If this does not hold the sludge age $t_{S,12}$ may have to be raised.

The use of the anoxic volume for nitrification at low temperatures implies built-in aeration installations in separated denitrification zones.

For aerobic co-stabilization of sludge a sludge age of t_S=25 days is required. It may be reduced to a sludge age of 20 days if the sludge is post-stabilized e.g. in sludge lagoons.

The daily sludge production WAS_d (kg dry solids per day) is calculated separately for carbon removal $WAS_{d,C}$ and phosphorus removal $WAS_{d,P}$:

$$WAS_d = WAS_{d,C} + WAS_{d,P} \tag{7}$$

With the daily dry weather flow, Q_d (m^3/d), the inlet BOD_5 concentration, $C_{BOD,0}$ (mg/l), and the inlet suspended solids concentration, X_0 (mg/l), $WAS_{d,C}$ becomes:

$$WAS_{d,C} = Q_d \cdot \frac{C_{BOD,0}}{1000} \cdot \left(0.75 + 0.6 \cdot \frac{X_0}{C_{BOD,0}} - \frac{(1-0.2) \cdot 0.75 \cdot t_S \cdot 0.17 \cdot 1.072^{(T-15)}}{1 + t_S \cdot 0.17 \cdot 1.072^{(T-15)}} \right) \tag{8}$$

The coefficients in Eq. (8) are according to Hartwig (1993). The difference compared to the value of $WAS_{d,C}$ of the 1991 issue of the guideline is neglectable.

$$WAS_{d,P} = Q_d \cdot (3 \cdot X_{P,BioP} + 6.8 \cdot X_{P,prec,Fe} + 5.3 \cdot X_{P,prec,Al})/1000 \tag{9}$$

It is assumed that per mg of biologically bound phosphorus 3 mg suspended solids are formed. The factors 6.8 and 5.3 for precipitation by Fe^{3+} and Al^{3+}, respectively, are stoichiometric factors. $X_{P,BioP}$ and $X_{P,prec}$ are in mg/l P.

The mass of sludge to be kept in the biological reactor M_{SS} (kg) at design temperature of e.g. 12ºC and finally the reactor volume become:

$$M_{SS} = WAS_d \cdot t_{S,12} \tag{10}$$

$$V = \frac{M_{SS}}{X_{AT}} \tag{11}$$

The suspended solids concentration in the biological reactor X_{AT} (kg/m^3), except for the step-feed nitrogen removal process, is equal to the concentration of the secondary settling tank inlet as estimated under settling tank design: X_{AT}=X_F. At the step-feed process the mean X_{AT} may be 15 to 20% higher than X_F, depending on the number of steps and the wastewater distribution.

The oxygen uptake is calculated separately for carbon removal and for nitrogen removal. One of the main changes of the guideline is the implementation of more realistic coefficients of oxygen uptake for carbon removal $OU_{d,C}$ (kg/d) (Hartwig, 1993):

$$OU_{d,C} + Q_d \cdot \frac{C_{BOD,0}}{1000} \cdot \left(0.56 + \frac{t_S \cdot 0.15 \cdot 1.072^{(T-15)}}{1 + t_S \cdot 0.17 \cdot 1.072^{(T-15)}}\right) \tag{12}$$

Since the coefficients were derived for COD/BOD_5=2.0 it is recommended to check the oxygen uptake at plants with a higher COD/BOD_5-ratio by a calculation based on COD.

The daily oxygen uptake for nitrification $OU_{d,N}$ (kg/d) and the daily oxygen equivalent from denitrification $OU_{d,D}$ (kg/d) become:

$$OU_{d,N} = Q_d \cdot 4.3 \cdot (S_{NO3,D} – S_{NO3,0} + S_{NO3,E})/1000 \tag{13}$$

$$OU_{d,D} = –Q_d \cdot 2.9 \cdot S_{NO3,D}/1000 \tag{14}$$

Considering a peaking factor of the oxygen uptake rate for carbon removal f_C and for nitrification f_N, respectively, the peak hourly oxygen uptake OU_h (kg/h) is obtained:

$$OU_h = \{f_C \cdot (OU_{d,C} - OU_{d,D}) + f_N \cdot OU_{d,N}\} /24 \tag{15}$$

The peaking factor for carbon removal can be determined only experimentally or by dynamic simulation. The peaking factor for nitrification may be calculated as the ratio of the hourly peak ammonia load and the daily mean ammonia load. If load data are not available the peaking factors may be taken from a table presented in the guideline. Since the peak for nitrification usually precedes the peak for carbon removal it is recommended to perform two calculations: one with f_C=1 and f_N=f_N and one with f_C=f_C and f_N=1; the higher value shall be used for the layout of the aeration installation.

Since the required oxygen transfer rate will be about 20% lower than calculated by the 1991 issue of the guideline it becomes important not to overestimate the oxygen transfer capacity of the aeration facility to be installed.

In deep aeration tanks with fine bubble diffused air aeration due to the low stripping rate of carbon dioxide the pH may drop to an inhibitory range. The guideline contains a table showing the pH calculated according to Nowak (1996) as function of the oxygen transfer efficiency and the remaining alkalinity.

COD based calculations

The incomming COD ($C_{COD,0}$) consists of the soluble COD ($S_{COD,0}$ determined by 0.45 µm membrane filtration) and the particulate COD ($X_{COD,0}$=$C_{COD,0}$– $S_{COD,0}$). $C_{COD,0}$ is furthermore divided into the biodegradable soluble COD ($S_{COD,bio}$)), the biodegradable particulate COD ($X_{COD,bio}$), the inert soluble COD ($S_{COD,inert,0}$=$S_{COD,inert,E}$~0.05 to 0.1 $C_{COD,0}$) and the inert particulate COD ($X_{COD,inert,0}$ ~0.2 to 0.35 $X_{COD,0}$. The biomass produced ($X_{COD,BM}$) is calculated by using Y=0.67 and b=0.17·1.072$^{(T-15)}$ in analogy to Henze *et al.* (1987). About 20% of the biomass lost by decay is assumed to remain as inert biomass ($X_{COD,BM,inert}$). The COD of the solids wasted ($X_{COD,WAS}$) becomes:

$$X_{COD,WAS} = X_{COD,inert,0} + (C_{COD,0} - S_{COD,inert,0} - X_{COD,inert,0}) \cdot Y \cdot \frac{1}{1 + b \cdot t_S} \\ +0.2 \cdot X_{COD,BM} \cdot t_S \cdot b \qquad (16)$$

The solids wasted may be calculated by assuming 1.45 mg COD/mg organic SS and an organic fraction of WAS solids of 80%. Influent inorganic suspended solids have to be added.

The oxygen uptake for COD removal OU (mg/l O_2) becomes:

$$OU = C_{COD,0} - S_{COD,inert,E} - X_{COD,WAS} \qquad (17)$$

The major problem of the application of the COD based calculation is that the soluble COD of the wastewater ($S_{COD,0}$) is not analyzed as a routine parameter.

Conclusions

The ATV design guideline A 131 is based on scientific knowledge and practical experience. It is a "static" design procedure for activated sludge plants. Diurnal load fluctuations as usually observed at municipal wastewater treatment plants with respect to the effluent ammonia and nitrate concentrations are considered by some safety factors. The oxygen uptake to lay-out the aeration equipment shall be calculated on the basis of the diurnal load fluctuation. If the diurnal load fluctuations are higher than usual it is recommended to check the results by dynamic simulation.

In case of pronounced seasonal fluctuations the design shall be checked for the respective loads. The result may be that the activated sludge tank volume has to be sized for the e.g. lower load at lower temperatures but the oxygen requirement may reflect the seasonal higher load in summer.

The final sedimentation tank in any case has to be designed for the highest flow rate entering the plant at storm periods. It is essential to harmonize the operation of storm water retention basins of the sewer system and the hydraulic capacity of the wastewater treatment plant because the performance of the biological reactor may deteriorate caused by extreme shock loads.

References

ATV (1991). Arbeitsblatt A 131: *Bemessung von einstufigen Belebungsanlagen ab 5000 Einwohnerwerten.* GFA, Hennef.

ATV (1999). Entwurf Arbeitsblatt A 131: *Bemessung von einstufigen Belebungsanlagen*. GFA, Hennef.

Hartwig, P. (1993). Beitrag zur Bemessung von Belebungsanlagen mit Stickstoff- und Phosphorelimination. Veröffentlichungen des Institutes für Siedlungswasserwirtschaft und Abfalltechnik der Universität Hannover. Heft 84.

Henze, M., Grady, C.P.L. Jr., Gujer, W., Marais, G.v.R. and Matsuo, T. (1987). *Activated Sludge Model No. 1*. IAWPRC Scientific and Technical Reports No. 1. IAWPRC, London.

Ludzack, F.J. and Ettinger, M.B. (1962). Controlling operation to minimize activated sludge effluent nitrogen. Journal WPCF 35, 920–931.

Matsché, N.F. (1977). Removal of nitrogen by simultaneous nitrification-denitrification in an activated sludge plant with mammoth rotor aeration. *Prog. Wat. Tech.*, **8**(4–5), 625–637.

Miyaji, Y., Iwasaki, M. and Serviga, Y. (1980). Biological nitrogen removal by step-feed process. *Prog. Wat. Tech.*, **12**(6), 193–202.

Nowak, O. (1996). Nitrifikation im Belebungsverfahren bei maflgebendem Industrieabwassereinflufl. Wiener Mitteilungen Wasser, Abwasser, Gewässer. Band 135.

Tholander, B. (1977). An example of design of activated sludge plants with denitrification. *Prog. Wat. Tech.*, **8**(4–5), 661–672.