GERMAN ATV-DVWK

RULES AND STANDARDS

STANDARD
ATV-DVWK-A 131E

Dimensioning of Single-Stage Activated Sludge Plants

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Notes for Users

This ATV-DVGW Standard is the result of honorary, technical-scientific/economic collaboration which has been achieved in accordance with the principles applicable therefore (statutes, rules of procedure of the ATV and ATV Standard ATV-A 400). For this, according to precedents, there exists an actual presumption that it is textually and technically correct and also generally recognised.

The application of this Standard is open to everyone. However, an obligation for application can arise from legal or administrative regulations, a contract or other legal reason.

This Standard is an important, however, not the sole source of information for correct solutions. With its application no one avoids responsibility for his own action or for the correct application in specific cases; this applies in particular for the correct handling of the margins described in the Standard.

Foreword

At the time of elaborating the previous issue of this ATV Standard (1988-90) there were only isolated activated sludge plants with nitrogen and phosphorus removal, from whose operating results information could be deduced for dimensioning and operation. Therefore, with many questions, one had to rely exclusively on the results of research. In the meantime, a large number of such facilities have been commissioned so that a wider database, also from practice, is available for a revision.

Compared with the issue of ATV Standard ATV-A 131 dated February 1991 the following important changes have been made:

- validity for activated sludge plants of any size (up to now ≥ 5,000 total number of inhabitants and population equivalents PT).
- the chapter on derivation of design flows and loads is taken out, since a separate ATV Standard is to be elaborated for all types of wastewater treatment processes.
- dimensioning temperature for nitrogen removal T = 12° C in accord with the requirements from Appendix 1 of The [German] Wastewater Ordinance (AbwV) (previously T = 10° C), under the assumption of a flexible design of the biological rector.
- integration of dimensioning for excess biological phosphorus removal.
- modification of the denitrification capacity.
- change of the determination of the required oxygen transfer.
- integration of the dimensioning of an aerobic selector.
- option for dimensioning on the basis of the chemical oxygen demand (COD).
- increase of the permitted sludge volume loading rate of secondary settling tanks.
- modification of the designation of partial depths and the determination of the depth of the thickening and sludge removal zone of secondary settling tanks.
- integration of the dimensioning of the sludge removal systems (scrapers) in secondary settling tanks.

Explanations on process technology are to be taken from the ATV Manuals “Biological and advanced wastewater treatment “[1] and “Mechanical wastewater treatment” [2]. The figures additionally mentioned in the text refer to the chapters of the manuals.

1 Area of Application

1.1 Preamble

The treatment of the stormwater in the sewer network and of wastewater in the wastewater treatment plant form one unit for the protection of surface waters. For the dimensioning of the wastewater treatment plant and the stormwater overflows the planning periods are to be matched to each other. The planning period should comprise not more than 25 years.

1.2 Objective

Using the dimensioning values recommended in this standard, for municipal wastewater one can, with single-stage activated sludge plants, meet the achievable minimum effluent requirements which correspond with or undercut the requirements of the [German] Wastewater Ordinance (AbwV) dated 02.09.99, Appendix 1, and the associated sampling regulations. If commercial or industrial wastewater with high fractions of slowly biodegradable and/or inert organic substances is discharged, a higher residual COD than with domestic wastewater can arise. The same applies for areas with low water consumption and a low infiltration rate, as then the inert COD concentration increases.

Technical regulations are drawn up for the selection of the most practical procedure for carbon, nitrogen and phosphorus removal, and for the dimensioning of the essential components and facilities of the plant. The selection and dimensioning of aeration equipment is not dealt with in this standard.

Since this standard is also applied outside Germany and because locally even stricter requirements can be set, it is not aimed exclusively at the observance of the effluent requirements laid down in Appendix 1 of the Wastewater Ordinance (AbwV).

In accord with the requirements under water law, the structural and operating requirements and the sensitivity of the surface waters the planning through parallel units, reserve equipment etc. is to be oriented towards an appropriately high operational safety.

A prerequisite for the secure function of the plant, planned in accordance with this standard, is that sufficiently qualified, trained and permanently technically supported operating personnel are employed and involved in the planning process, comp. ATV Advisory Leaflet ATV-M 271 „Personalbedarf für den Betrieb kommunaler Kläranlagen“ [Personnel requirement for the operation of municipal sewage treatment plants].

1.3 Scope

This standard basically applies for the dimensioning of single-stage activated sludge plants. Due to the peculiarities of smaller sewage treatment plants attention is drawn to ATV Standards ATV-A 122E and ATV-A 126E as well as DIN 4261.

The standard applies for wastewater which essentially originates from households or from plants which serve commercial or agricultural purposes, insofar as the harmfulness of this wastewater can be reduced by means of biological processes with the same success as with wastewater from households.

2 Symbols

A_{ST}	m^2	Surface area of secondary settling tanks
a	-	Number of scraper blades in circular settling tanks
$B_{d,BOD}$	kg/d	Daily BOD_5 load
$B_{d,XXX}$	kg/d	Daily load for another parameter
$B_{R,BOD}$	$kg/(m^3 \cdot d)$	BOD_5 volume loading rate
$B_{R,XXX}$	$kg/(m^3 \cdot d)$	Volume loading rate with another parameter
$B_{SS,BOD}$	$kg/(kg \cdot d)$	BOD_5 sludge loading rate
$B_{SS,XXX}$	$kg/(kg \cdot d)$	Sludge loading rate with another parameter
b	d^{-1}	Decay coefficient
C_S	mg/l	Dissolved oxygen saturation concentration dependent on the temperature and partial pressure
C_X	mg/l	Dissolved oxygen concentration in aeration tanks (DO)
D_{ST}	m	Diameter of secondary settling tanks
DSV	l/m^3	Diluted sludge volume, 30 minutes settled (to be determined, if SV_{30} is higher than 250 L/m^3, what generally is the case)
F_T	-	Temperature factor for endogenous respiration
f_C	-	Peak factor for carbon respiration
f_N	-	Peak factor for ammonium oxidation
f_{SR}	-	Sludge removal factor, dependent on the type of sludge scraper
h_1	m	Depth of the clear water zone in secondary settling tanks
h_2	m	Depth of the separation zone / return flow zone in secondary settling tanks
h_3	m	Depth of the density flow and storage zone in secondary settling tanks
h_4	m	Depth of the sludge thickening and removal zone in secondary settling tanks
h_{In}	m	Depth of the centre of the inlet aperture (below water surface) of secondary settling tanks
h_{SR}	m	Height of a scraper blade or a scraper beam
h_{tot}	m	Total water depth in the secondary settling tank
L_{FS}	m	Length of a flight scraper in a rectangular tank ($L_{FS} \sim L_{ST}$)
L_{RW}	m	Length of the runway of a scraper bridge in rectangular settling tanks ($L_{RW} \sim L_{ST}$)
L_{SL}	m	Length of the sludge layer moved by a scraper blade in a rectangular settling tank ($L_{SL} \sim 15 \cdot h_{SR}$)
L_{SR}	m	Scraper blade or scraper beam length in rectangular secondary settling tanks ($L_{SR} \approx W_{ST}$)
L_{ST}	m	Length of rectangular secondary settling tanks

$M_{SS,AT}$	kg	Mass of suspended solids in the biological reactor / aeration tank
OC	kg/h	Oxygen transfer of an aeration facility in clean water with C_x = 0, T = 20° C and air pressure p = 1013 hPa
αOC	kg/h	Oxygen transfer of an aeration facility in activated sludge with C_x = 0, T = 20° C and air pressure p = 1013 hPa
$OU_{C,BOD}$	kg/kg	Oxygen uptake for carbon removal, referred to BOD_5
$OU_{d,C}$	kg/d	Daily oxygen uptake for carbon removal
$OU_{d,D}$	kg/d	Daily oxygen uptake for carbon removal which is covered by denitrification
$OU_{d,N}$	kg/d	Daily oxygen uptake for nitrification
OU_h	kg/h	Oxygen uptake rate (hourly)
PT_{XXX}	I	Total number of inhabitants and population equivalents referred to the parameters XXX, e.g. BOD_5, COD etc.
Q	m^3/h	Inflow rate, flow rate, throughflow rate
$Q_{DW,d}$	m^3/d	Daily wastewater inflow with dry weather
$Q_{DW,h}$	m^3/h	Hourly dry weather flow rate as 2 hr mean
$Q_{WW,h}$	m^3/h	Dimensioning peak flow rate with wet weather from combined and separate sewer systems
Q_{RS}	m^3/h	Return (activated) sludge flow rate
Q_{IR}	m^3/h	Internal recirculation flow rate at pre-anoxic zone denitrification process
Q_{RC}	m^3/h	Total recirculation flow rate ($Q_{RS} + Q_{IR}$) at pre-anoxic zone denitrification process
Q_{Short}	m^3/h	Short circuit sludge flow rate in secondary settling tanks
Q_{SR}	m^3/h	Sludge removal flow rate
$Q_{WS,d}$	m^3/d	Daily waste (activated) sludge flow rate
q_A	m/h	Surface overflow rate of secondary settling tanks
q_{SV}	$l/(m^2 \cdot d)$	Sludge volume surface loading rate of secondary settling tanks
RC	-	Total recirculation ratio at pre-anoxic zone denitrification process (RC = $Q_{RC}/Q_{h,DW}$)
RS	-	Return sludge ratio (RS = $Q_{RS}/Q_{h,DW}$ or $Q_{RS}/Q_{h,WW}$)
SF	-	Safety factor for nitrification
SP_d	kg/d	Daily waste activated sludge production (solids)
$SP_{d,C}$	kg/d	Daily sludge production from carbon removal
$SP_{d,P}$	kg/d	Daily sludge production from phosphorus removal
$SS_{C,BOD5}$	kg/kg	Sludge production from carbon removal referred to BOD_5
SS_{AT}	kg/m^3	Suspended solids concentration in the biological reactor / aeration tank (MLSS)
$SS_{AT,Step}$	kg/m^3	Average suspended solids concentration in the biological reactor with step-feed denitrification ($SS_{AT,Step} > SS_{EAT}$)
SS_{BS}	kg/m^3	Suspended solids concentration in the bottom sludge of secondary settling tanks

SS_{EAT}	kg/m^3	Suspended solids concentration in the effluent of the biological reactor / aeration tank (usually $SS_{EAT} = SS_{AT}$)
SS_{RS}	kg/m^3	Suspended solids concentration of the return (activated) sludge
SS_{WS}	kg/m^3	Suspended solids concentration of the waste (activated) sludge
SVI	l/kg	Sludge volume index
T	°C	Temperature in the biological reactor / aeration tank
T_{ER}	°C	Temperature in the biological reactor at which the effluent requirements for nitrogen have to be met
T_{Dim}	°C	Temperature in the biological reactor / aeration tank upon which dimensioning is based
T_W	°C	Temperature in the biological reactor in winter, $T_W < T_{Dim}$
t_D	h,d	Duration of denitrification phase with intermittent process
t_N	h,d	Duration of the nitrification phase with intermittent process
t_R	h,d	Retention period (e.g. $t_R = V_{AT} : Q_{h,DW}$)
t_s	h	Time for raising and lowering the scraper blade
t_{SR}	h	Sludge removal interval (Period of time for one loop of a scraper)
t_{SS}	d	Sludge age referred to V_{AT}
$t_{SS,dim}$	d	Sludge age upon which dimensioning is based
$t_{SS,aerob}$	d	Aerobic sludge age referred to V_N
$t_{SS,aerob,dim}$	d	Aerobic sludge age upon which dimensioning for nitrification is based
t_T	h	Cycle time with intermittent process ($t_T = t_D + t_N$)
t_{Th}	h	Thickening time of the sludge in the secondary settling tank
V_{AT}	m^3	Volume of the biological reactor / aeration tank
V_{BioP}	m^3	Volume of an anaerobic mixing tank for biological phosphorus removal
V_D	m^3	Volume of the biological reactor used for denitrification
V_N	m^3	Volume of the biological reactor used for nitrification
V_{Sel}	m^3	Volume of an aerobic selector
V_{ST}	m^3	Volume of the secondary settling tank
v_{ret}	m/h	Return velocity of the scraper bridge
v_{SR}	m/h	Scraper bridge velocity (with circular tanks at the periphery)
W_{ST}	m	Width of rectangular secondary settling tanks
Y	mg/mg	Yield factor (mg formed biomass (COD) per mg biodegradable COD)
α	-	Quotient of oxygen transfer in activated sludge and in clean water

Chemical parameters and concentrations:

C_{XXX}	mg/l	Concentration of the parameter XXX in the homogenised sample
S_{XXX}	mg/l	Concentration of the parameter XXX in the filtered sample (0.45 µm membrane filter)
X_{XXX}	mg/l	Concentration of the filter residue (solids), $X_{XXX} = C_{XXX} - S_{XXX}$

Frequently used parameters:

C_{BOD}	mg/l	Concentration of BOD_5 in the homogenised sample
C_{COD}	mg/l	Concentration of COD in the homogenised sample
$C_{COD,deg}$	mg/l	Concentration of biodegradable COD
C_N	mg/l	Concentration of total nitrogen in the homogenised sample as N
C_P	mg/l	Concentration of phosphorus in the homogenised sample as P
C_{TKN}	mg/l	Concentration of Kjeldahl nitrogen in the homogenised sample ($C_{TKN} = C_{orgN} + S_{NH4}$)
C_{orgN}	mg/l	Concentration of organic nitrogen in the homogenised sample ($C_{orgN} = C_{TKN} - S_{NH4}$ or $C_{orgN} = C_N - S_{NH4} - S_{NO3} - S_{NO2}$)
S_{ALK}	mmol/l	Alkalinity
S_{BOD}	mg/l	Concentration of BOD_5 in the 0.45 µm filtered sample
S_{COD}	mg/l	Concentration of COD in the 0.45 µm filtered sample
$S_{COD,deg}$	mg/l	Concentration of dissolved, biodegradable COD
$S_{COD,inert}$	mg/l	Concentration of dissolved, inert COD
$S_{COD,ext}$	mg/l	Concentration of dissolved COD added as external carbon for the improvement of denitrification
S_{inorgN}	mg/l	Concentration of inorganic nitrogen ($S_{inorgN} = S_{NH4} + S_{NO3} + S_{NO2}$)
S_{NH4}	mg/l	Concentration of ammonium nitrogen in the filtered sample as N
S_{NO3}	mg/l	Concentration of nitrate nitrogen in the filtered sample as N
S_{NO2}	mg/l	Concentration of nitrite nitrogen in the filtered sample as N
$S_{NO3,D}$	mg/l	Concentration of nitrate nitrogen to be denitrified
$S_{NO3,D,ext}$	mg/l	Concentration of nitrate nitrogen to be denitrified with external carbon
$S_{NH4,N}$	mg/l	Concentration of ammonium nitrogen to be nitrified
S_{PO4}	mg/l	Concentration of phosphate as P (dissolved)
$X_{COD,BM}$	mg/l	Concentration of COD of the biomass
$X_{COD,deg}$	mg/l	Concentration of particulate, biodegradable COD
$X_{COD,inert}$	mg/l	Concentration of particulate, inert COD
$X_{orgN,BM}$	mg/l	Concentration of organic nitrogen embedded in the biomass
$X_{P,BM}$	mg/l	Concentration of phosphorus embedded in the biomass
$X_{P,Prec}$	mg/l	Concentration of phosphorus removed by simultaneous precipitation

$X_{P,BioP}$	mg/l	Concentration of phosphorus removed with biological excess phosphorus removal process
X_{SS}	mg/l	Concentration of suspended solids of wastewater (0.45 μm membrane filters after drying at 105° C)
$X_{org,SS}$	mg/l	Concentration of organic suspended solids of wastewater
$X_{inorgSS}$	mg/l	Concentration of inorganic suspended solids of wastewater

Indices on the location or purpose of the sampling (always last)

I	Sample from influent to the wastewater treatment plant
IAT	Sample from the influent to the biological reactor, if applicable of the influent to the anaerobic mixing tank, e.g. $C_{COD,IAT}$
EAT	Sample from the effluent of the biological reactor, e.g. $S_{NO3,EAT}$
EDT	Sample from the effluent of the denitrification tank, e.g. $S_{NO3,EDT}$
ENT	Sample from the effluent of the nitrification tank, e.g. $S_{NH4,\ ENT}$
EST	Sample from the effluent of the secondary settling tank, e.g. $C_{BOD,EST}$, $X_{SS,EST}$
WS	Sample from the waste (activated) sludge
RS	Sample from the return (activated) sludge
ER	Effluent requirement with a defined sampling procedure

3 Process Description and Procedure of Dimensioning

3.1 General

The activated sludge process is a unit process comprising the biological reactor (activated sludge tank) with the aeration equipment and the secondary settling tank, both connected by the return sludge recirculation.

The settling behaviour of the activated sludge, characterised by the sludge volume index (SVI), in combination with the mixed liquor suspended solids concentration (SS_{AT}), influences the size of the secondary settling tanks and biological reactors. Both the characteristics of the wastewater as well as the configuration of the biological reactor and the treatment target influence the sludge volume index. Biological reactors, which are to be considered as completely mixed tanks, usually lead to higher sludge volume indices and tend rather to the development of filamentous bacterial growth than tanks with a concentration gradient, which are such which, for example, are formed as a cascade or in which a plug flow exists. With wastewater having a high fraction of readily biodegradable organic matter, the inclusion of an upstream selector is helpful; upstream anaerobic mixing tanks for excess biological phosphorus removal do also have such a selector effect, see Fig. 1. The figure serves the nomenclature and does not imply that either an aerobic tank or a selector has to be an integral part of an activated sludge plant. However, it is pointed out that, using selectors, the growth of filamentous organisms is not controllable in all cases.

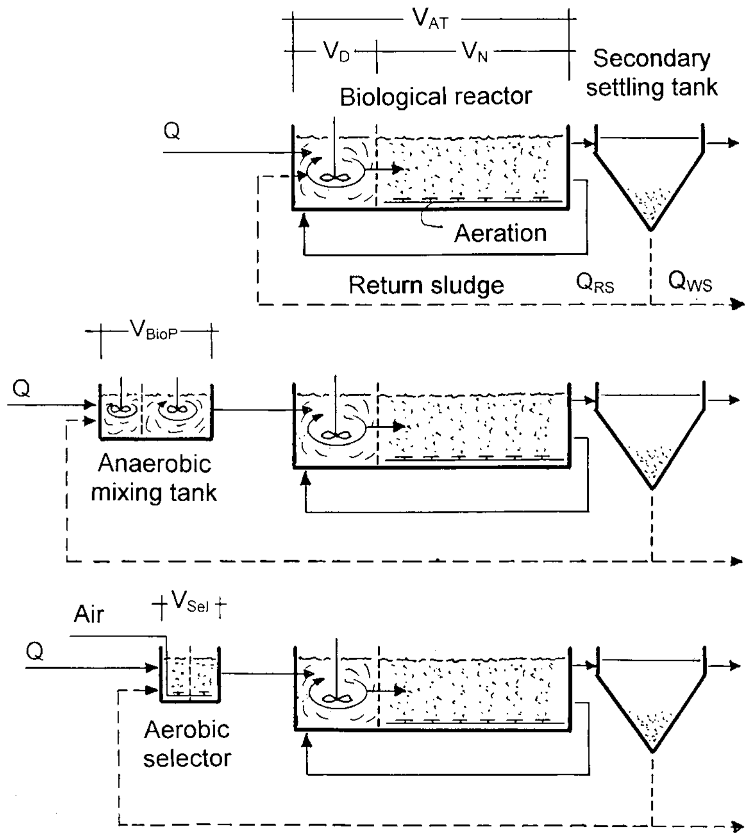

Fig.1: Flow diagram for the nomenclature of an activated sludge plant for nitrogen removal without and with an upstream anaerobic mixing tank for biological phosphorus removal or an aerobic selector

In the place of the pre-anoxic zone denitrification process shown in Fig. 1, almost all other processes for nitrogen removal and also aeration tanks, which serve only the removal of organic carbon, can be combined with an aerobic selector or an anaerobic mixing tank. The volume of an aerobic selector (V_{Sel}) or of an anaerobic mixing tank for phosphorus removal (V_{BioP}) is considered not to be a part of the biological reactor (V_{BB}). In plants, which are designed only to carbon removal, the volume of an aerobic selector can be considered as part of the aeration tank.

Relevant for the dimensioning of the biological reactor is the sludge age (t_{SS}), which corresponds approximately with the retention period of a sludge floc in the biological reactor. It is defined as the quotient of the mass of suspended (dry) solids in the biological reactor ($V_{AT} \cdot SS_{AT}$) and the daily mass of dry solids of waste activated sludge.

If the biological reactor has anoxic zones for denitrification (V_D), the aerobic sludge age ($t_{SS,aerob}$) is defined as the quotient of the dry solid mass of the sludge in the aerobic part of the biological reactor ($V_N = V_{AT} - V_D$) and the daily mass of waste activated sludge.

The residual pollution of the effluent of the secondary settling tank is, in a large part, caused by dissolved and colloidal matter and in part by suspended (activated sludge) solids. This is dependent on the efficiency of the secondary settling tank. A suspended solids concentration of 1 mg/l dry solids in the secondary settling tank effluent increases the concentration of

C_{BOD} by 0.3 to 1.0 mg/l
C_{COD} by 0.8 to 1.4 mg/l
C_N by 0.08 to 0.1 mg/l
C_P by 0.02 to over 0.04 mg/l

3.2 Biological Reactor

The treatment of the wastewater by the activated sludge process, with regard to process technology, operating and economic aspects, places the following requirements on the biological reactor (aeration tank):

– sufficient enrichment of the biomass, measured simplified as the mixed liquor suspended solids concentration of the activated sludge (SS_{AT});

– sufficient oxygen transfer to cover the oxygen uptake and its control to match the different operating and loading conditions;

– sufficiently mixing in order to prevent a permanent settling of sludge on the tank bottom; as a rule ensured in aeration tanks through aeration, if required supported by mixing facilities; as guidance values for the bottom velocity in areas outside bottom installed diffused air aeration facilities, 0.15 m/s with light sludge and 0.30 m/s with heavy sludge may be assumed. In anaerobic or anoxic mixing tanks the mixing is ensured by the mixing facilities. Depending on the tank size and shape, power inputs of 1 to 5 W/m^3 are normal.

– no nuisances caused by odours, aerosols, noise and vibrations.

For nitrogen removal various reactor constructions and operating modes are possible (Fig. 2); these can be characterised as follows (comp. [1] 5.2.5 and 5.3.2), whereby the above listed requirements are always to be observed:

– pre-anoxic zone denitrification process: wastewater, return sludge and internal recirculation flow are mixed in the denitrification tank. Both denitrification tanks and nitrification tanks can be formed as cascades. To increase the operating flexibility, viewed in the flow direction, the last parts of the denitrification tank can also be capable of being aerated. Internal recirculation is to be reduced to the absolutely necessary in order to minimise the negative interferences of high loads of dissolved oxygen on denitrification.

– step-feed denitrification process: two or more biological reactors, each with pre-anoxic zone or simultaneous denitrification, are streamed one after the other. The wastewater is apportioned and fed respectively to the denitrification tanks. As a rule, through this, internal recirculation is dispensed with. High oxygen contents at the transfer from nitrification tank to the following denitrification tank prejudices the denitrification. With regard to nitrogen removal, the process is equivalent to the pre-anoxic zone denitrification process. Due to the separate feed of the wastewater, the concentration of mixed liquor in the first tank is higher than in the effluent to the secondary settling tank, comp. [1] 5.2.5.4.

– simultaneous denitrification process: in practice only to realise in circulating flow (carousel) tanks. The circulating water flows through the denitrification and nitrification zones in the tank. One can consider simultaneous denitrification to be a type of pre-anoxic zone denitrification with a high internal recirculation ratio. An automatic control of the aeration, for example according to the nitrate content, the ammonium content, the break in the redox potential (redox break) or the oxygen content respectively the oxygen uptake rate, is necessary. With regard to the dilution, circulation tanks approximate to completely mixed tanks.

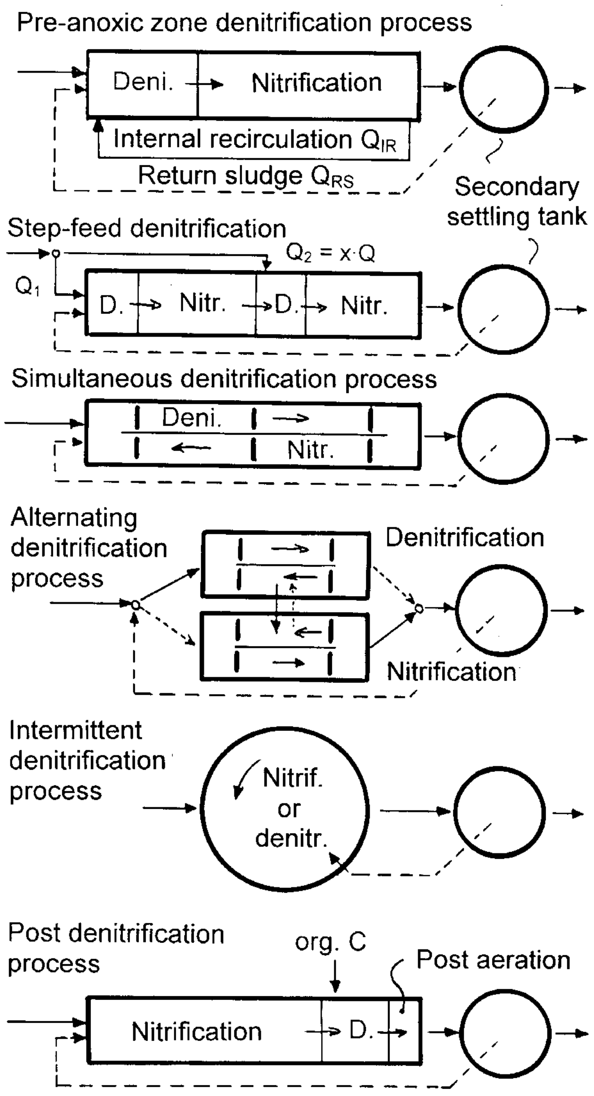

Fig. 2: Nitrogen removal procedures

– alternating denitrification process (BioDenitro): two respectively intermittently aerated tanks are charged one after the other, whereby water flows from the charged unaerated tank into the other aerated tank and from there to the secondary settling tank. The duration of charging as well as the duration of the denitrification and the nitrification phases are, as a rule, timer controlled. High oxygen contents at the end of the nitrification phase prejudice the denitrification. The mixing behaviour lies between that of completely mixed and plug flow tanks.

– intermittent denitrification process: the nitrification and denitrification phases alternate in time in one reactor. The duration of a phase can be timer controlled or by automatic control, for example according to the nitrate content, the ammonium content, the break in the redox potential or the oxygen uptake rate. High oxygen contents at the end of the nitrification phase prejudice the denitrification. The reactors for intermittent denitrification are to be considered as completely mixed tanks.

– post denitrification process: the process is employed if the wastewater has a very low C/N ratio so that the addition of external carbon is unavoidable. The denitrification tank is downstream from the nitrification tank; for safety reasons a post-aeration tank follows.

***[Adendum (NOT in original German text):** In nitrification tanks and during nitrification phases of intermittent processes aeration normally is automatically controlled in order to achieve a sufficient dissolved oxygen concentration (DO)**]**.*

In addition to the above-named procedures there exists a series of in part patented special processes for nitrogen removal, comp. [1], 5.2.5.

Sequencing batch activated sludge plants (SBR plants) are also suitable for nitrogen removal. Further details can be found in ATV Advisory Leaflet ATV-M 210 and in [1], 5.3.3.

A significant excess biological phosphorus removal is observed in many activated sludge plants for nitrogen removal even without an upstream anaerobic tank.

For excess biological phosphorus removal upstream of each single biological reactor or of a group of biological reactors an anaerobic mixing tank for wastewater and return sludge is placed (comp. [1], 5.2.6 and 5.3.2), Fig. 1. The effectiveness can be increased if the anaerobic tank is designed as a cascade, as then nitrate contained in the return sludge is removed in one part and in the other part completely anaerobic conditions are achieved. Attention is drawn to [1], 5.2.6 for special procedures. Facilities for simultaneous phosphorus precipitation are provided in most plants with excess biological phosphorus removal. The precipitant dosing should as far as possible be automatically controlled whereby a control system in the outflow of the aeration tank is preferred.

Excess biological phosphorus removal is also possible in activated sludge plants which are designed only for carbon elimination, if the sludge age t_{SS} is at least 2 to 3 days.

3.3 Secondary Settling Tank

Secondary settling tanks have the main task of separating the activated sludge from the biologically treated wastewater .

The loading capacity of an activated sludge plant is determined substantially by the concentration of suspended solids (SS_{AT}) of the activated sludge and the volume of the aeration tank. The concentration of suspended solids depends essentially on the functional capability of the secondary settling tanks with fluctuating hydraulic feeding, the sludge volume index and the sludge removal as well as the return sludge ratio and the waste sludge removal.

Dimensioning, design and equipping of secondary settling tanks must be carried out that the following tasks can be met:

– separation of the activated sludge from treated wastewater by settling;
– thickening and removal of the settled activated sludge for recirculation to the biological reactor (aeration tank);
– intermediate storage of activated sludge which, as a result of increased inflow rates at stormwater periods ($Q_{WW,h}$), is expelled from the aeration tank.

The settling process in the secondary settling tank is influenced by the flocculation process in the inlet zone, the hydraulic conditions in the secondary settling tank (design of the inlet and outlet, density currents) the return sludge ratio and the sludge removal procedure. The settled sludge is concentrated in the sludge layer on the tank bottom. The thickening achieved therein is dependent on the sludge properties (SVI), the depth of the sludge layer, the thickening time and the type of the sludge removal system.

With stormwater inflow activated sludge is relocated increasingly from the aeration tank into the secondary settling tank. The secondary settling tank must then be able to store the sludge expelled from the aeration tank. For this a sufficiently large storage volume, an efficient sludge removal system and appropriately dimensioned sludge return facilities (e.g. pumps) are required.

With regard to the method of function one differentiates between horizontal and vertical flow secondary settling tanks. Circular and rectangular tanks are differentiated according to design. The settled and thickened sludge, so far as it does not flow automatically into the sludge hopper, is transported by blade or flight scrapers or is removed directly using suction facilities.

3.4 Procedure of Dimensioning

The dimensioning of activated sludge plants takes place iteratively, as many factors influence each other mutually, comp. Fig. 3. The calculation path given below practically represents one calculation run after which it can be necessary to repeat the calculations with new assumptions.

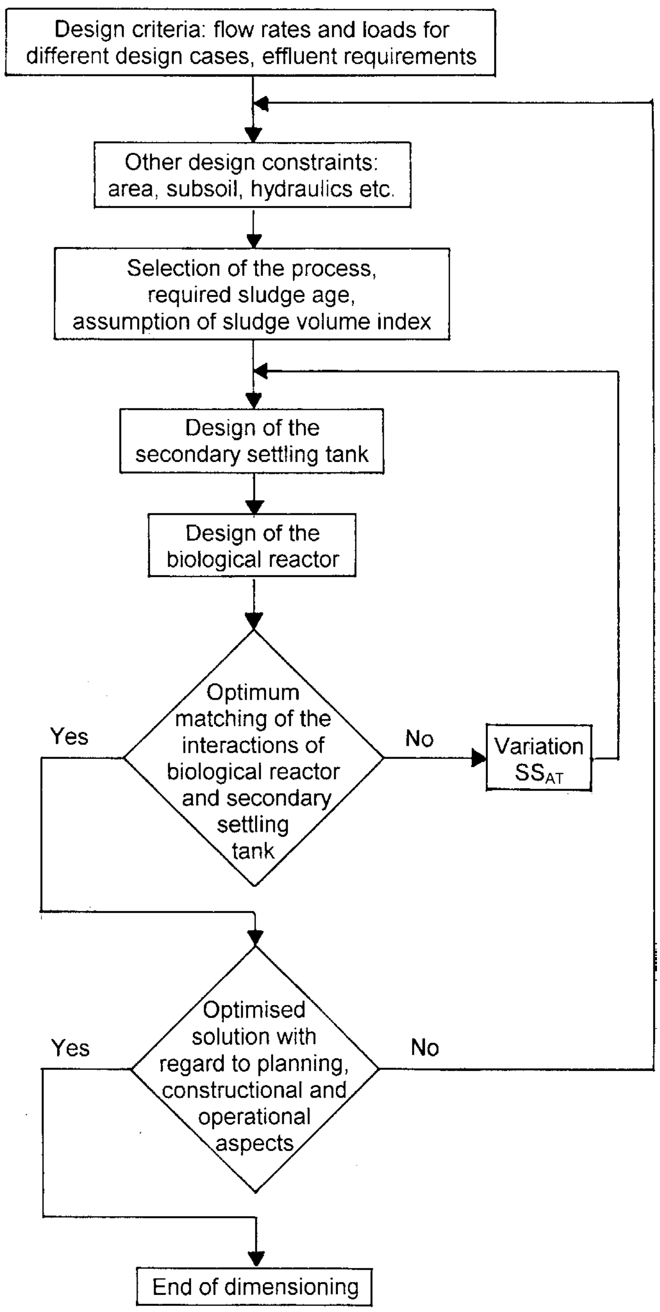

Fig. 3 Sequence of planning and dimensioning

The following steps are recommended:

1 determination of the dimensioning capacity of the plant and the relevant flows and loads to the biological reactor, comp. Chap. 4.

2 selection of the process: if nitrogen removal is required, it has to be decided which process for nitrification/denitrification is to be employed. In addition it is to be determined whether an aerobic selector for the improvement of the settling characteristics or an anaerobic mixing tank for biological excess phosphorus removal is to be placed upstream.

3 determination of the necessary safety factor (SF) taking into account the dimensioning capacity of the plant and, in case, the measured diurnal load fluctuations. For plants which are designed for nitrification only, the sludge age ($t_{SS,aerob,dim}$) is to be determined taking into account the dimensioning temperature. Both are omitted with aerobic sludge stabilisation.

4 with plants for nitrogen removal the mass of the nitrate to be denitrified is to be determined by means of a nitrogen balance. If not a percentage of nitrogen removal is to be maintained but rather a concentration value, the influent concentration is of great influence; if the concentration in a random sample is to be met (e.g. qualified random sample in accordance with the Wastewater Ordinance in Germany), this must be taken specially into account with the dimensioning.

5 taking into account the selected denitrification process the necessary proportion of the denitrification volume to the biological reactor volume (V_D/V_{AT}) is to be determined. The sludge age ($t_{SS,dim}$) is to be calculated accordingly. For combined aerobic sludge stabilisation the sludge age is to be selected, if appropriate in accordance with the relevant wastewater temperature.

6 selection of the sludge volume index taking into account the composition of the wastewater, the configuration and the mixing characteristics of the biological reactor as well as, if selected, an aerobic selector or anaerobic mixing tank.

7 selection of the sludge thickening time (t_{Th}) in the secondary settling tank dependent on the biological process selected and determination of the concentration of (dry) suspended solids in the bottom sludge (SS_{BS}) as function of SVI and t_{Th}.

8 determination of the return sludge suspended solids concentration (SS_{RS}) from the achievable concentration of suspended solids in the bottom sludge and the dilution of the sludge removal stream dependent on the selected sludge removal system.

9 selection of the return sludge ratio (RS) and estimation of the permissible suspended solids concentration of the activated sludge in the biological reactor (SS_{AT}).
The mixed liquor suspended solids concentration of the activated sludge influences the volumes of biological reactors and secondary settling tanks in the opposite sense. It is to be noted that the volume of the biological reactor reduces with increasing SS_{AT} while, with increasing $SS_{AT,}$ the surface area of the secondary settling tanks and, in addition, the depth become greater.

10 determination of the surface area of the secondary settling tank (A_{ST}) from the permissible surface overflow rate (q_A) or the sludge volume loading rate (q_{SV}).

11 determination of the depth of the secondary settling tank from partial depths for the functional zones and other specifications.

12 verification of the selected thickening time by the sludge removal (scraper) performance, prerequisite is that the dimensions of the secondary settling tank are laid down.

13 determination of the waste sludge production (SP_d), if required taking into account waste sludge from phosphorus removal and the possibly dosed external carbon for denitrification.

14 calculation of the required mass of solids in the biological rector ($M_{SS,AT}$) for the selected sludge age.

15 calculation of the volume of the biological reactor.

16 if required, dimensioning of an anaerobic mixing tank for biological phosphorus removal.

17 calculation of the necessary internal recirculation flow rate for pre-anoxic zone denitrification or the cycle time with intermittent denitrification processes.

18 determination of the relevant oxygen consumption for the design of the aeration facility.

19 checking of the remaining alkalinity and/or the necessity for dosing lye taking into account consumption and gain in alkalinity from ammonification, nitrification, denitrification and phosphate precipitation as well as the oxygen utilisation and diffuser depth (The latter only to determine the pH in the biological reactor).

20 if required, dimensioning of an aerobic selector for the improvement of the settling properties of the activated sludge.

The dimensioning parameters can be laid down on the basis of scientific model concepts and supported by experience or, in part, can be derived from experiments carried out on site.

4 Dimensioning Flows and Loads

4.1 Loading with Wastewater

The dimensioning value of the wastewater treatment plant $B_{d,BOD,I}$ in kg/d BOD_5 (raw) for the classification into the Size Class in accordance with Appendix 1 to the [German] Wastewater Ordinance and for the determination of the dimensioning capacity of the plant in the Assessment under Water Law is derived from the BOD_5 load in the influent to the wastewater treatment plant which is undercut on 85 % of the dry weather days, plus a planned capacity reserve. If the dimensioning capacity is determined based on the number of connected inhabitants, the inhabitant-specific BOD_5 load for raw wastewater from Table 1 is to be used.

In principle it applies that sewer system and sewage treatment plant are to be operated for the same wastewater effluent and influent.

For the dimensioning, the following important numerical values are required from the influent to the biological reactor, if applicable with the inclusion of the return flows from sludge treatment (comp. 4.2):

– relevant lowest and highest wastewater temperature. Determination from the curve of the 2-week mean over two to three years.

– relevant organic load ($B_{d,BOD}$ $B_{d,COD}$), the relevant load of suspended solids ($B_{d,SS}$) and of phosphorus ($B_{d,P}$) for the determination of the sludge production and thus the calculation of the volume of the aeration tank for the dimensioning temperature.

– relevant organic load and nitrogen load for the design of the aeration facility for (as a rule) the highest relevant temperature.

– relevant concentration of nitrogen (C_N) and the related concentration of organic substances (C_{BOD}, C_{COD}) for the determination of the nitrate to be denitrified.

– relevant concentration of phosphorus (C_P) for the determination of the phosphorus to be removed.

– maximum inflow rate with dry weather $Q_{DW,h}$ (m^3/h) for the design of the anaerobic mixing tank and the internal recirculation flow rate.

– dimensioning inflow rate $Q_{WW,h}$ (m^3/h) for the design of the secondary settling tanks.

Daily loads can only be calculated on the basis of volumetric- or flow proportional 24 hr composite samples and the related daily inflow. The relevant loads are to be determined on the basis of measurements on arbitrary days, i.e. with the inclusion of wet weather days.

If an annual graph indicates periodical fluctuations of the organic loads or/and the ratio of organic load to nitrogen load, several loading cases are to be investigated.

The relevant concentrations are to be determined using the relevant loads and the associated daily wastewater inflows. The relevant loads for periods with relevant wastewater temperatures, are made up as mean value of a period of time, which corresponds with the sludge age. Simplified for nitrification and denitrification two-week means and, for sludge stabilisation, four-week means can be made up. If, in the absence of sufficient sampling density (at least four utilisable daily loads per week), weekly means cannot be made up, then the loads which are undercut on 85 % of the days are relevant, whereby at least 40 load values should be used.

If the data are insufficient or the expense of investigations, for example with small plants, are in no relation to the use, loads and concentrations can be determined on the basis of connected inhabitants plus industrial-commercial and other loads.

Details on the determination of the relevant loads and concentrations are to be found in the Standard ATV-DVWK-A 198 [in preparation] "Unification and derivation of dimensioning values for wastewater systems" [3].

If the relevant loads have to be estimated using the connected inhabitants, the values in Table 1 can be used. The estimate of the associated wastewater inflow is to be undertaken in accordance with the ATV-DVWK Standard [3]. Until publication of that Standard the determination of the wastewater inflow can be undertaken in accordance with ATV Standard ATV-A 131 (1991) [not translated into English].

Table 1: Inhabitant-specific loads in g/(I·d), which are undercut on 85 % of the days, without taking into account sludge liquor

Parameter	Raw wastewater	Flow time in the primary settling stage with $Q_{h,DW}$	
		0.5 to 1.0 h	1.5 to 2.0 h
BOD_5	60	45	40
COD	120	90	80
DS	70	35	25
TKN	11	10	10
P	1.8	1.6	1.6

Deliberate wastewater investigations and determinations of loads over two to four weeks cannot, as a rule, be used directly for dimensioning as one cannot be sure of having recorded the relevant time period. They are, however, practical for the supplementing of the existing database. With such investigations the inflows associated with the sampling intervals are always to be recorded. Thus the diurnal TKN curves for the determination of the f_N value (comp. 5.2.8) can be recorded. Seldom analysed values such as, for example, the suspended solids concentration ($X_{SS,IAT}$) or the alkalinity ($S_{ALK,IAT}$) can thus be covered. The loading of internal return flows e.g. from sludge treatment should also be recorded within the scope of such investigations.

4.2 Loading with Sludge Liquor and External Sludge

Water from the thickening and dewatering of (anaerobic) digested sludge contains ammonium in high concentrations. It can be assumed that 50 % of the organic nitrogen introduced into the sludge digester is released as ammonium nitrogen. If sludge liquor is produced for a few hours daily only or only on odd days weekly, an intermediate storage for dosed input is necessary.

The return loading with phosphorus and organic matter (BOD_5 and COD) is, as a rule, small from dewatering of digested sludge. Therefore a return loading may not be added, for example, globally as a percentage to all loads from the wastewater.

In sludge silos for aerobic stabilised sludge, as a rule, more or less anaerobic processes occur. With this, ammonium can be released and redissolution of phosphorus is possible, if excess biological phosphorus removal is applied. In order to minimise impairment of the biological treatment

– sludge liquor should be drawn off regularly in small quantities

– when dewatering the silo content filtrate or centrate should be collected in silos of a similar size and be fed to the inlet over a long period of time.

If external sludge (sludge from other sewage treatment plants, faecal sludge or similar) is discharged then an intermediate storage can be sensible, in order to make a dosed input possible.

5 Dimensioning of the Biological Reactor

5.1 Dimensioning on the Basis of Pilot Experiments

Pilot testing using pilot plants or a part of the full size plant serve for the examination of a process concept and of the model parameters under practice-oriented conditions.

The pilot plants for this are to be established at least on a semi-technical scale and are to be operated under practice-oriented operating conditions for not shorter than half a year with the inclusion of the cold season. One can carry out a weakest-point analysis beforehand with the aid of dynamic simulation. From this one can gather valuable information for planning of the test runs.

Through such investigations the dimensioning is, as a rule, more correct and often costs can be saved. Using the results an improved basis for the dynamic simulation of operational conditions not recorded during the experiments is then also created.

Some of the dimensioning parameters given under Sect. 3.4 can be determined with this, these are, for example:

– the sludge production and the necessary sludge age.

– the practical subdivision of the biological reactor (anaerobic, anoxic and aerobic), if required for the various seasons and/or loading conditions.

– the oxygen consumption and the requirements for automatic control of the oxygen transfer; for this the oxygen uptake rate has to be measured frequently.

– the dissolved residual COD ($S_{COD,EST}$)

5.2 Dimensioning on the Basis of Experience

5.2.1 Required Sludge Age

5.2.1.1 Plants without Nitrification

Activated sludge plants without nitrification are dimensioned for sludge ages of four to five days, comp. Table 2.

Table 2: Dimensioning sludge age in days dependent on the treatment target and the temperature as well as the plant size (intermediate values are to be estimated)

Treatment target	Size of the plant $B_{d,BOD,I}$ Up to 1,200 kg/d		Over 6,000 kg/d	
Dimensioning temperature	10° C	12° C	10° C	12° C
Without nitrification	5		4	
With nitrification	10	8.2	8	6.6
With nitrogen removal V_D/V_{AT} = 0.2 0.3 0.4 0.5	 12.5 14.3 16.7 20.0	 10.3 11.7 13.7 16.4	 10.0 11.4 13.3 16.0	 8.3 9.4 11.0 13.2
Sludge stabilisation incl. nitrogen removal	25		Not recommended	

5.2.1.2 Plants with Nitrification

The (aerobic) dimensioning sludge age to be maintained for nitrification is:

$$t_{SS,aerob,\dim} = SF \cdot 3.4 \cdot 1.103^{(15-T)} \qquad [d] \qquad (5\text{-}1)$$

The value of 3.4 is made up from the reciprocal of the maximum (net) growth rate of the ammonium oxidants (nitrosomonas) at 15° C (2.13 d) and a factor of 1.6. Through the latter it is ensured that, with sufficient oxygen transfer and no other negative influence factors, enough nitrificants can be developed or held in the activated sludge, (comp. [1] 5.2.4). With a sludge age of 2.13 d (15° C), nitrificants cannot accumulate.

Using the safety factor (SF) the following are taken into account:

– variations of the maximum growth rate caused by certain substances in the wastewater, short-term temperature variations or/and pH shifts.

– the mean effluent concentration of the ammonium.

– the effect of variations of the influent nitrogen loads on the variations of the effluent ammonia concentration.

Based on all previous experience it is recommended, for municipal plants with a dimensioning capacity up to $B_{d,BOD,I}$ = 1,200 kg/d (20,000 PT), to reckon with SF = 1.8 due to the more pronounced influent load fluctuation and for $B_{d,BOD,I} \geq$ 6,000 kg/d (100,000 PT) with SF = 1.45. With this, the effluent concentration on average, can be held at $S_{NH4,EST}$ = 1.0 mg/l, so long as no negative influencing of the maximum growth rate of the nitrificants exists.

If, with plants with $B_{d,BOD,I}$ < 6,000 kg/d, the **measured** f_N value lies below 1.8 (comp. 5.2.8), SF can be reduced down to 1.45.

If a buffering tank for daily balancing of the load is planned the safety factor shall not be assumed smaller than SF = 1.45.

If, in winter, the temperature in the outflow of the biological reactor sinks below the temperature at which the effluent requirement for ammonium has to be held (T_{ER}), the dimensioning value in Eqn. 5-1 T_{dim} = (T_{ER} - 2) is to be applied, in order to achieve a stable nitrification at the control temperature. It is proposed that, for the control temperature of T_{ER} = 12° C in dependence on the size of the plant taking into account the above-given safety factor to select the following dimensioning sludge ages:

Plants up to $B_{d,BOD,I}$ = 1,200 kg/d
$t_{SS,aerob,dim}$ = 10 d

Plants above $B_{d,BOD,I}$ = 6,000 kg/d
$t_{SS,aerob,dim}$ = 8 d

These values are given in Table 2. Intermediate values are to be interpolated.

If the wastewater temperature is always higher than the control temperature, the lowest two-week mean of the temperature can be selected as the dimensioning temperature.

In order to limit the heavy consumption of alkalinity (comp. 5.2.9) through nitrification, it is recommended, for operational reasons, to plan a partial denitrification, comp. 5.2.1.3.

5.2.1.3 Plants with Nitrification and Denitrification

Prerequisite for nitrogen elimination is a secure nitrification, comp. 5.2.1.2.

For nitrification and denitrification the dimensioning sludge age results as follows:

$$t_{SS,\dim} = t_{SS,aerob} \cdot \frac{1}{1-(V_D / V_{AT})} \qquad [d] \qquad (5\text{-}2)$$

With Eqn. 5-1:

$$t_{SS,aerob,\dim} = SF \cdot 3.4 \cdot 1.103^{(15-T)} \cdot \frac{1}{1-(V_D / V_{AT})} \qquad [d] \qquad (5\text{-}3)$$

Attention is drawn to Sect. 5.2.2 for the calculation of V_D/V_{AT}.

In Eqn. 5-3 the temperature to be applied as dimensioning temperature is that at which nitrogen elimination is required (T_{dim} = T_{ER}); thus, in accordance with the Wastewater Ordinance in Germany, T_{dim} = T_{ER} = 12° C.

For the wastewater temperatures which, as a rule, in winter are lower than 12° C, proof is to be furnished that, with the lowest two-week mean of the temperature, the nitrification does not break down. For this, maintaining the dimensioning sludge age, the portion V_D/V_{AT} for the lower temperature (T_W) is calculated according to Eqn. 5-4.

If there are no acceptable measured values available for the wastewater temperature, the temperature t_{ER}, reduced by 2° to 4° C, should be applied in Eqn. 5-4 for T_W. (2 C, if a cooling of the wastewater below 10° C in the two-week mean is not to be expected and 4° C, if in extreme situations heavier cooling is to be reckoned with).

If, with the lower temperatures, the organic loading ($B_{d,BOD,I}$) is an other than the one on which dimensioning is based, the actual sludge age should be applied in Eqn. 5-4 instead of $t_{SS,dim}$.

$$V_D / V_{AT} = \frac{SF \cdot 3.4 \cdot 1.103^{(15-T_W)}}{t_{SS,\dim}} \qquad [-] \qquad (5\text{-}4)$$

This proof assumes a flexible design of the biological reactor, whereby the denitrification zone has to be reducible in favour of the nitrification zone. A possibly available anaerobic mixing tank can be included in the volume V_D at pre-anoxic zone denitrification, if the internal recirculation is appropriately designed.

If, according to Eqn. 5-4, there is a negative value for V_D/V_{AT}, then V_D/V_{AT} = 0 is applied and the safety factor is to be calculated using Eqn. 5-4. It can be lowered down to SF = 1.2; otherwise the reactor volume is to be increased.

Should a dimensioning temperature below 12° C be required, one proceeds accordingly. There is no experience available about the dimensioning of plants for a temperature below 8° C.

In every case it is to be verified whether the remaining alkalinity is sufficient, comp. Sect. 5.2.9.

If the effluent requirement for ammonium nitrogen is set with $S_{NH4,ER}$ < 10 mg/l or the influent loads are subject to very high variations, even in dry weather, and the monitoring takes place on a random sample or a 2 h composite sample, the safety factor is to be increased or a verification with the aid of dynamic simulation is be carried out. This calls for the measurement of the appropriate diurnal load fluctuations.

5.2.1.4 Plants with Aerobic Sludge Stabilisation

The dimensioning sludge age of plants, which are to be dimensioned for aerobic sludge (co-) stabilisation and nitrification, must be $t_{SS,dim} \geq 20$ d.

If deliberate denitrification is also required the sludge age must be $t_{SS,dim} \geq 25$ d. If the temperature in the biological reactor in the two-week mean is always higher than 12° C, the sludge age can be reduced in accordance with Eqn. 5-5.

$$t_{SS,\dim} \geq 25 \cdot 1.072^{(12-T)} \qquad [d] \qquad (5\text{-}5)$$

If the organic loads in the warm season are higher than those in the cold season, the required mass of sludge $M_{SS,AT}$ (comp. 5.2.6) have to be determined separately for both cases using Eqn. 5.5. The greater mass of sludge is relevant for the biological reactor volume.

If sludge ponds or tanks with at least a storage duration of one year of the liquid sludge for anaerobic post-stabilisation are available, the sludge age, even if deliberate denitrification is demanded, can be lowered to $t_{SS,dim} = 20$ d.

The calculation of the nitrate to be denitrified and the volume fraction V_D/V_{AT} takes place according to Sect. 5.2.2. V_D/V_{AT} has no influence on the sludge age but rather serves, for example with intermittent denitrification, for the calculation of the oxygen transfer.

5.2.2 Determination of the Proportion of the Reactor Volume for Denitrification

The daily average nitrate concentration to be denitrified results as follows:

$$S_{NO3,D} = C_{N,IAT} - S_{orgN,EST} - S_{NH4,EST} - S_{NO3,EST} - X_{orgN,BM} \qquad [mg/l] \qquad (5\text{-}6)$$

As influent nitrogen concentration ($C_{N,IAT}$) the relevant value determined for T = 12° C is to be applied. If, during the year, at the times of higher temperatures, higher $C_{N,IAT}/C_{COD,IAT}$ ratios have been determined, in case several types of load are to be considered.

The influent nitrate concentration ($S_{NO3,IAT}$) is, in general, negligibly small. With greater infiltration rates (groundwater containing nitrate) or with inflows from certain commercial and industrial plants, it can be necessary to take account of $S_{NO3,IAT}$ in $C_{N,IAT}$.

At plants with anaerobic sludge digestion and mechanical dewatering at the site, the nitrogen of the sludge liquor must be contained in the inflow concentration ($C_{N,IAT}$) if no separate sludge liquor treatment takes place, comp. Sect. 4.2. The concentration of organic nitrogen in the effluent can be set as $S_{orgN,EST} = 2$ mg/l. With the inflow of certain commercial wastewater the concentration can be higher. To be on the safe side the ammonium content in the effluent for dimensioning is, as a rule, assumed as $S_{NH4,EST} = 0$. The nitrogen incorporated in the biomass is taken into account simplified as $X_{orgN,BM} = 0.04$ to $0.05 \cdot C_{BOD,IAT}$ or 0.02 to $0.025 \cdot C_{BOD,IAT}$.

The relevant effluent concentration of the nitrate is to be applied as daily average, If, as in Germany, the monitoring takes place by means of random grab or 2 hour composite samples, a significantly smaller concentration than the effluent requirement for inorganic nitrogen ($S_{inorgN,ER}$) has to be selected. It is practical to set $S_{NO3,EST} = 0.8$ to $0.6 \cdot S_{inorgN,ER}$, whereby the smaller value applies for plants with high variations of the influent load.

With the relevant BOD_5 of the inflow to the biological reactor (or to the anaerobic mixing tank) one obtains the ratio $S_{NO3,D}/C_{BOD,IAT}$, which gives the necessary denitrification capacity.

For simultaneous and intermittent denitrification processes the following calculation of V_D/V_{AT} can be applied, comp. [1], 5.2.5.3:

$$\frac{S_{NO3,D}}{C_{BOD,IAT}} = \frac{0.75 \cdot OU_{C,BOD}}{2.9} \cdot \frac{V_D}{V_{AT}} \qquad \text{[mg N/mg BOD}_5\text{]} \qquad (5\text{-}7)$$

*[**Addendum (NOT in original German text):** Eqn. 5-7 is derived from a mass balance of oxygen in the denitrification zone of a completely mixed biological reactor.]*

$$\frac{Q_d \cdot 2.9 \cdot S_{NO3,D}}{1000} = V_D \cdot 0.75 \cdot \frac{OU_{d,C}}{V_{AT}} \qquad \textit{[kg/d]}$$

*The left hand side represents the oxygen provided by the daily nitrate load to be denitrified. The right side shows the daily uptake of oxygen in the denitrification zone. The factor 0.75 indicates an overall lower uptake rate of nitrate compared to the uptake rate of dissolved oxygen**]**.*

$OU_{C,BOD}$ is to be determined in accordance with Eqn. 5-24 for the dimensioning sludge age and the dimensioning temperature or is to be taken from Table 7. For the temperature range from 10° to 12° C the values calculated using Eqn. 5-7 are listed in Table 3.

For pre-anoxic zone denitrification process and comparable processes, at which only a small part of the readily biodegradable organic matter is lost for the denitrification, the empirical values listed in Table 3, which match the tendency towards the theoretically derivable values, apply, comp. [1], Fig. 5.2.5-3. Prerequisite is that in all influents to the denitrification zone the dissolved oxygen content is kept at less than 2 mg/l.

Table 3: Standard values for the dimensioning of denitrification for dry weather at temperatures from 10° to 12° C and common conditions (kg nitrate nitrogen to be denitrified per kg influent BOD_5)

V_D/V_{AT}	$S_{NO3,D}/C_{BOD,IAT}$	
	Pre-anoxic zone denitrification and comparable 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

For the temperature range from 10° to 12° C it is recommended to use for dimensioning the values for the denitrification capacity in Table 3. Denitrification volumes smaller than V_D/V_{AT} = 0.2 and greater than V_D/V_{AT} = 0.5 are not recommended for dimensioning.

The denitrification capacity with the alternating denitrification process can be assumed to be the average between pre-anoxic zone and intermittent denitrification.

For temperatures above 12° C the denitrification capacity can be increased by ca. 1 % per 1° C.

If the dimensioning or re-calculation takes place on the basis of COD, one can reckon with $S_{NO3,D}/C_{COD,IAT} = 0.5 \cdot (S_{NO3,D}/C_{BOD,IAT.})$.

With re-calculation for a value of $V_D/V_{AT} = 0.1$, one can reckon with $S_{NO3,D}/C_{BOD,IAT} = 0.08$ for pre-anoxic zone denitrification and $S_{NO3,D}/C_{BOD,IAT} = 0.03$ for simultaneous and intermittent denitrification. If, by re-calculation a value of $V_D/V_{AT} < 0.1$ is obtained then $S_{NO3,D}/C_{BOD,IAT} = 0$ is to be set.

If the required denitrification capacity is larger than $S_{NO3,D}/C_{BOD} = 0.15$, then a further increase of V_D/V_{AT} is not recommended. It is to be investigated whether a volume reduction or partial by-passing of the primary settling tank and/or, if applicable, a separate sludge treatment are conducive to meeting the target. An alternative is to carry out the planning for the addition of external carbon. The construction of the appropriate facilities should, however, first be undertaken, if secured operational experience is available.

The requirement for external carbon is ca. 5 kg COD per kg of nitrate nitrogen to be denitrified. With this one obtains the average increase of the COD as:

$$S_{COD,Ext} = 5 \cdot S_{NO3,D,Ext} \qquad \text{[mg/l]} \qquad (5\text{-}8)$$

The COD of commercial carbon compounds can be taken from Table 4. For other sources of carbon the COD and, if necessary, the denitrification capacity, are to be determined in advance. It is pointed out that methanol is only suitable for a long-term application as special denitrificants have to be grown.

Table 4: Characteristics of external carbon sources

Parameter	Unit	Methanol	Ethanol	Acetic acid
Density	kg/m^3	790	780	1,060
COD	kg/kg	1.50	2.09	1.07
COD	kg/L	1.185	1.630	1.135

5.2.3 Phosphorus Removal

Phosphorus removal can take place alone through simultaneous precipitation, through excess biological phosphorus removal, as a rule combined with simultaneous precipitation, and through pre- or post precipitation (comp. [1], 5.2.6 and 7.4).

Anaerobic mixing tanks for biological phosphorus removal are to be dimensioned for a minimum contact time of 0.5 to 0.75 hours, referred to the maximum dry weather inflow and the return sludge flow ($Q_{DW,h} + Q_{RS}$). The degree of the biological phosphorus removal depends, other than on the contact time, to a large extent on the ratio of the concentration of readily biodegradable organic matter to the concentration of phosphorus. If, in winter, the anaerobic volume is used for denitrification, then during this period a lower biological excess phosphorus removal will establish.

For the determination of the phosphate to be precipitated a phosphorus balance, if necessary for different types of load, is to be drawn up:

$$X_{P,Prec} = C_{P,IAT} - C_{P,EST} - X_{P,BM} - X_{P,BioP} \quad \text{[mg/l]} \qquad (5\text{-}9)$$

$C_{P,IAT}$ is the concentration of the total phosphorus in the influent to the biological reactor. The effluent concentration ($C_{P,EST}$) is to be selected in agreement with the effluent requirement for phosphorus ($C_{P,ER}$), e.g. $C_{P,EST}$ = 0.6 to 0.7 $C_{P,ER}$. The phosphorus necessary for the build-up heterotrophic biomass ($X_{P,BM}$) can be set as 0.01 $C_{BOD,IAT}$ or 0.005 $C_{COD,IAT}$ respectively. With normal municipal wastewater one can assume the following for the excess biological phosphorus removal ($X_{P,BioP}$):

– $X_{P,BioP}$ = 0.01 to 0.015 · $C_{BOD,IAT}$ or 0.005 to 0.007 $C_{COD,IAT}$ respectively with upstream anaerobic tanks.

– if, with lower temperatures, $S_{NO3,EST}$ increases to ≥ 15 mg/l, it can be assumed: $X_{P,BioP}$ = 0.005 to 0.01 $C_{BOD,IAT}$ or 0.0025 to 0.005 · $C_{COD,IAT}$ respectively with upstream anaerobic tanks.

– in plants with pre-anoxic zone denitrification or step-feed denitrification, but without anaerobic tanks, an excess biological phosphorus removal of $X_{P,BioP}$ ≤ 0.005 $C_{BOD,IAT}$ or 0.002 $C_{COD,IAT}$ respectively can be assumed.

– if, at low temperatures, the internal recirculation of pre-anoxic zone denitrification is discharged into the anaerobic tank, one can reckon with $X_{P,BioP}$ ≤ 0.005 $C_{BOD,IAT}$ or 0.002 $C_{COD,IAT}$ respectively.

The mean precipitant requirement can be calculated using 1.5 mol Me^{3+}/mol $X_{P,Prec}$. Converted the following requirement values are obtained:

Precipitation using iron	2.7 kg Fe/kg P_{Prec}
Precipitation using aluminium	1.3 kg Al/kg P_{Prec}

For simultaneous precipitation using lime, as a rule, milk of lime is dosed into the influent to the secondary settling tank in order to raise the pH and through this to bring about precipitation. In the first instance, the requirement for lime depends on the alkalinity. Tests are in any case recommended, comp. ATV Standard ATV-A 202 [not yet available in English].

For control values of $C_{P,ER}$ < 1.0 mg/l, e.g. $C_{P,ER}$ = 0.8 mg/l in the qualified random sample, single-stage activated sludge plants cannot be dimensioned. In practice, however, values of $C_{P,EST}$ = < 1.0 mg/l can be achieved under favourable conditions.

5.2.4 Determination of the Sludge Production

The sludge produced in an activated sludge plant is made up of organic matter resulting from degradation and stored solid matter as well as sludge resulting from phosphorus removal:

$$SP_d = SP_{d,C} + SP_{d,P} \quad \text{[kg/d]} \qquad (5\text{-}10)$$

The relationship of sludge production and sludge age can be written as follows:

$$t_{SS} = \frac{M_{SS,AT}}{SP_d} = \frac{V_{AT} \cdot SS_{AT}}{SP_d} = \frac{V_{AT} \cdot SS_{AT}}{Q_{WS,d} \cdot SS_{WS} + Q_d \cdot X_{SS,EST}} \quad \text{[d]} \qquad (5\text{-}11)$$

As the load of filterable matter in the effluent of the secondary settling tank ($Q_d \cdot X_{SS,EST}$) is, as a rule, negligible, the sludge production (SP_d) can be assumed to be the same as the daily waste sludge ($Q_{WS,d} \cdot SS_{WS}$).

For the calculation of the sludge production from carbon removal the following empirical equation using the Hartwig coefficients can be used (comp. [1], 5.2.8.2):

$$SP_{d,C} = B_{d,BOD} \cdot (0.75 + 0.6 \cdot \frac{X_{SS,IAT}}{C_{BOD,IAT}} - \frac{(1-0.2) \cdot 0.17 \cdot 0.75\, t_{SS} \cdot F_T}{1+0.17 \cdot t_{SS} \cdot F_T}) \quad \text{[kg/d]} \qquad (5\text{-}12)$$

The temperature factor (F_T) for the endogenous respiration is:

$$F_T = 1.072^{(T-15)} \quad \text{[-]} \qquad (5\text{-}13)$$

If external carbon has to be dosed regularly for the improvement of denitrification, then with $S_{COD,Ext} \geq 10$ mg/l ($S_{NO3,D,Ext} \geq 2$ mg/l) simplified in Eqn. 5-12, $B_{d,BOD}$ is to be increased by the value $Q_d \cdot 0.5 \cdot S_{COD,Ext}/1000$ and in Eqn. 5-12 as well as in Table 5 $C_{BOD,IAT}$ by the value $0.5 \cdot S_{COD,Ext}$. With $S_{COD,Ext} \leq 10$ mg/l the additional sludge production is ignored.

The values in Table 5 are calculated and averaged using Eqn. 5-12 for T = 10° C and 12° C.

Table 5: Specific sludge production $SP_{C,BOD}$ [kg SS/kg BOD_5] at 10° to 12° C

$X_{SS,IAT}/C_{BOD,IAT}$	Sludge age in days					
	4	8	10	15	20	25
0.4	0.79	0.69	0.65	0.59	0.56	0.53
0.6	0.91	0.81	0.77	0.71	0.68	0.65
0.8	1.03	0.93	0.89	0.83	0.80	0.77
1.0	1.15	1.05	1.01	0.95	0.92	0.89
1.2	1.27	1.17	1.13	1.07	1.04	1.01

The sludge production from the phosphorus removal is made up of the solid matter from the excess biological phosphorus removal and that from simultaneous precipitation.

For the excess biological phosphorus removal one can reckon with 3 g SS per g biologically removed phosphorus. The solids yield from simultaneous precipitation is dependent on the type of precipitant and the amount of dosing, comp. Sect. 5.2.3. One should reckon with a sludge production of 2.5 kg SS per kg dosed iron and 4 kg SS per kg dosed aluminium. The total sludge production resulting from phosphorus removal ($SP_{d,P}$) thus results as follows:

$$SP_{d,P} = Q_d \cdot (3 \cdot X_{P,BioP} + 6.8 \cdot X_{P,Prec,Fe} + 5.3 \cdot X_{P,Prec,Al})/1000 \quad \text{[kg/d]} \qquad (5\text{-}14)$$

If lime is used for precipitation the sludge production is 1.35 kg SS per kg calcium hydroxide ($Ca(OH)_2$); see also ATV Standard ATV-A 202.

5.2.5 Assumption of the Sludge Volume Index and the Mixed Liquor Suspended Solids Concentration

The sludge volume index depends on the composition of the wastewater and the mixing characteristics of the aeration tank. A high fraction of readily biodegradable organic matter, as are contained in some commercial and industrial wastewater, can lead to higher sludge volume indices.

The correct assumption of the sludge volume index is of particular significance for dimensioning. If solely the expansion of the secondary settling tank is to be planned, without biological process modification, the sludge volume index for dimensioning can be based on the operating records for the critical season or, alternatively, as the value undercut on 85 % of the days. However, even if biological process modifications are planned, the operating records together with the values in Table 6 are helpful for the estimation of the sludge volume index. If, in the past, sludge volume indices of SVI > 180 l/m^3 have been observed, measures for reduction should be taken.

*[**Addendum (NOT in original German text):** If the sludge volume after half an hour settling exceeds 250 ml/l the mixed liquor has to be diluted with final effluent so that a sludge volume between 100 and 250 ml/l is measured. Taking into account the dilution ratio the diluted sludge volume DSV is obtained.]*

Table 6: Standard values for the sludge volume index

Treatment target	SVI (l/kg) Industrial/commercial wastewater influence	
	Favourable	Unfavourable
Without nitrification	100 - 150	120 - 180
Nitrification (and denitrification)	100 - 150	120 - 180
Sludge stabilisation	75 - 120	100 - 150

If no usable data are available, the values listed in Table 6 are recommended for dimensioning taking into account critical operating conditions.

The respectively lower values for the sludge volume index (SVI) can be applied, if

– primary settling is dispensed with,

– a selector or an anaerobic mixing tank is placed upstream,

– the biological reactor is designed as a cascade (plug flow).

The concentration of mixed liquor suspended solids (SS_{AT}) is determined in the process of dimensioning the secondary settling tank. SS_{AT} can be taken from Fig. 4 for a pre-dimensioning of the biological reactor.

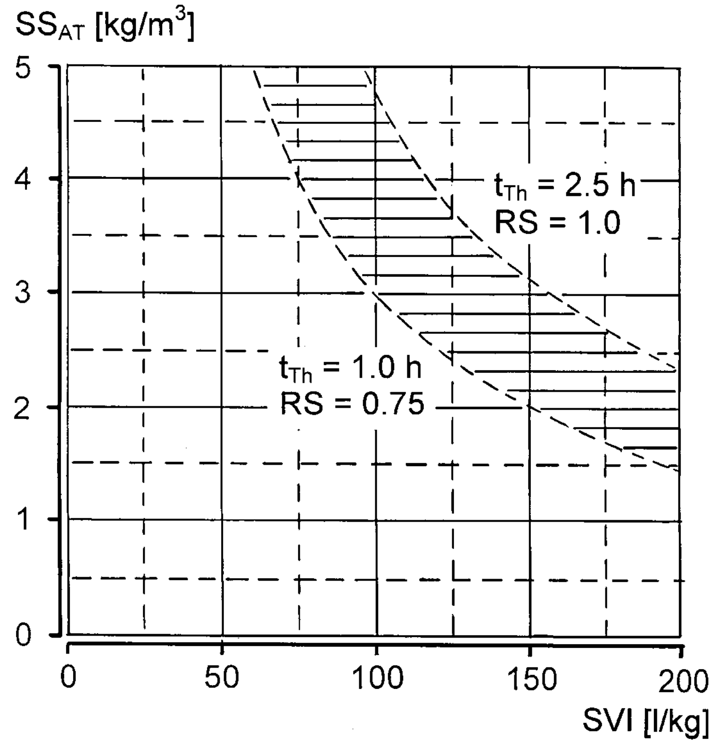

Fig. 4: Approximate values for the mixed liquor suspended solids concentration in the biological reactor dependent on the sludge volume index for $SS_{RS} = 0.7 \cdot SS_{BS}$

5.2.6 Volume of the Biological Rector

According to Eqn. 5-11 the required mass of suspended solids in the biological reactor is:

$$M_{SS,AT} = t_{SS,Dim} \cdot SP_d \qquad [kg] \qquad (5\text{-}15)$$

The volume of the biological reactor is obtained as follows:

$$V_{AT} = \frac{M_{SS,AT}}{SS_{AT}} \qquad [m^3] \qquad (5\text{-}16)$$

As comparative figures the BOD_5 volume loading rate (B_R) and the sludge loading rate (B_{SS}) can be calculated:

$$B_R = \frac{B_{d,BOD}}{V_{AT}} \qquad [kg\ BOD_5/(m^3 \cdot d)] \qquad (5\text{-}17)$$

$$B_{SS} = \frac{B_R}{SS_{AT}} \qquad [kg\ BOD_5/(kg\ SS \cdot d)] \qquad (5\text{-}18)$$

With tanks with step-feed denitrification process in Eqns. 5-16 and 5-18 in place of SS_{AT} $SS_{AT,Step}$ is to be applied. Thereby is $SS_{AT,Step} > SS_{EAT}$ or SS_{AT}, comp. [1], 5.2.5.4.

*[**Addendum (NOT in original German text):** Assume a step-feed process with units of equal sizes operated with RS = 1 and a feed distribution to reach similar sludge loading rates in all*

denitrification tanks. Then for a two step process $SS_{AT,Step}$ ~ 1.14 · SS_{EAT} and for a three step process $SS_{AT,Step}$ ~ 1.20 · SS_{EAT} holds.]

5.2.7 Required Recirculation and Cycle Time

The necessary total recirculation flow ratio (RC) for pre-anoxic zone denitrification results using $S_{NH4,N}$, the ammonium nitrogen concentration to be nitrified, as follows (comp. [1], 5.2.5.4):

$$RC = \frac{S_{NH4,N}}{S_{NO3,EST}} - 1 \qquad [-] \tag{5-19}$$

The following applies:

$$RC = \frac{Q_{RS}}{Q_{DW,h}} + \frac{Q_{IR}}{Q_{DW,h}} \qquad [-] \tag{5-20}$$

RC is determined using Eqn. 5-19, and the internal recirculation Q_{IR} is obtained using Eqn. 5-20. The maximum possible efficiency of denitrification is :

$$\eta_D \leq 1 - \frac{1}{1 + RC} \qquad [-] \tag{5-21}$$

With step-feed denitrification the efficiency is determined via the fraction (x) of the load fed to the last denitrification tank; if necessary an internal recirculation is to be taken into account. The following applies without internal recirculation (comp. [1], 5.2.5.4):

$$\eta_D = 1 - \frac{x}{1 + RS} \qquad [-] \tag{5-22}$$

With an intermittent denitrification process the cycle duration ($t_T = t_N + t_D$) can be estimated as follows (comp. [1], 5.2.5.4):

$$t_T = t_R \cdot \frac{S_{NO3,EST}}{S_{NH4,N}} \qquad [\text{h or d}] \tag{5-23}$$

The retention time $t_R = V_{AT}/Q_{h,DW}$ and the cycle time (t_T) have the same unit. A cycle time of less than 2 hours is not recommended.

5.2.8 Oxygen Transfer

The oxygen uptake is made up of the consumption for carbon removal (including the endogenous respiration) and, if necessary, the requirement for nitrification as well as the saving of oxygen from denitrification, comp. [1], 5.2.8.3.

For carbon removal the following approach, using the Hartwig coefficients, is applied, comp. [1], 5.2.8.3. With this the values in Table 7 were also calculated:

$$OU_{d,C} = B_{d,BOD} \cdot \left(0.56 + \frac{0.15 \cdot t_{SS} \cdot F_T}{1 + 0.17 \cdot t_{SS} \cdot F_T}\right) \qquad [\text{kg } O_2/\text{d}] \tag{5-24}$$

Dosed external carbon is not taken into account for oxygen utilisation, as it can be assumed that this is respired using nitrate.

The coefficients of Eqn. 5-24 apply for $C_{COD,IAT}/C_{BOD;IAT} \leq 2.2$. If a higher ratio has been found through measurements, it is necessary to determine the oxygen consumption for the design of the aeration facility with the aid of the COD, comp. appendix.

For nitrification the oxygen consumption is assumed to be 4.3 kg O_2 per kg oxidised nitrogen taking into account the metabolism of the nitrificants, comp. [1], 5.2.4.1. With denitrification one reckons for carbon removal with 2.9 kg O_2 per kg denitrified nitrate nitrogen:

$$OU_{d,N} = Q_d \cdot 4.3 \cdot (S_{NO3,D} - S_{NO3,IAT} + S_{NO3,EST}) / 1000 \qquad [\text{kg } O_2/\text{d}] \qquad (5\text{-}25)$$

$$OU_{d,D} = Q_d \cdot 2.9 \cdot S_{NO3,D} / 1000 \qquad [\text{kg } O_2/\text{d}] \qquad (5\text{-}26)$$

The oxygen uptake rate for the daily peak (OU_h) is obtained through:

$$OU_h = \frac{f_C \cdot (OU_{d,C} - OU_{d,D}) + f_N \cdot OU_{d,N}}{24} \qquad [\text{kg } O_2/\text{h}] \qquad (5\text{-}27)$$

The peak factor f_C represents the ratio of the oxygen uptake rate for carbon removal in the peak hour to the average daily oxygen uptake rate. Due to the equalisation effect of the hydrolysis of the solid matter this is not the ratio of the appropriate BOD_5 loads. Details for the calculation, comp. [1], 5.2.8.3. The peak factor f_N is equivalent to the ratio of the TKN load in the 2 h peak to the 24 h average load.

Table 7: Specific oxygen consumption $OU_{C,BOD}$ [kg O_2/kg BOD_5], valid for $C_{COD,IAT}/C_{BOD,IAT} \leq 2.2$

T° C	Sludge age in days					
	4	8	10	15	20	25
10	0.85	0.99	1.04	1.13	1.18	1.22
12	0.87	1.02	1.07	1.15	1.21	1.24
15	0.92	1.07	1.12	1.19	1.24	1.27
18	0.96	1.11	1.16	1.23	1.27	1.30
20	0.99	1.14	1.18	1.25	1.29	1.32

As the peak oxygen uptake rate for nitrification, as a rule, occurs before the appearance of the peak oxygen uptake rate for carbon removal, two calculations using Eqn. 5-27 are to be carried out, one with f_C = 1 and the determined/assumed f_N value, and one with f_N = 1 and the assumed/determined f_C value. The higher value of OU_h is relevant. With normal inflow conditions f_C and f_N can be taken from Table 8.

Table 8: Peak factors for the oxygen uptake rate (to cover the 2 h peaks compared with the 24 h average, if no measurements are available)

	Sludge age in d

	Sludge age in d					
	4	6	8	10	15	25
f_C	1.3	1.25	1.2	1.2	1.15	1.1
f_N for $B_{d,BOD,I} \leq 1200$ kg/d	-	-	-	2.5	2.0	1.5
f_N for $B_{d,BOD,I} > 6000$ kg/d			2.0	1.8	1.5	-

The necessary oxygen transfer for continuously aerated tanks then results as:

$$req.\,\alpha OC = \frac{C_S}{C_S - C_X} \cdot OV_h \qquad [\text{kg } O_2/\text{h}] \qquad (5\text{-}28)$$

For tanks, which are aerated intermittently, the aeration-free times are to be taken into account. The following applies:

$$req.\,\alpha OC = \frac{C_S}{C_S - C_X} \cdot OV_h \cdot \frac{1}{1 - V_D / V_{AT}} \qquad [\text{kg } O_2/\text{h}] \qquad (5\text{-}29)$$

The dissolved oxygen concentration (DO) in the aerated part of the aeration tank is to be applied for the dimensioning of the aeration facility using C_X = 2 mg/l. For circulating flow tanks with surface aerators, one can reckon with C_X = 0.5 mg/l for simultaneous denitrification due to the saw-tooth shaped profile of the dissolved oxygen concentration around the tank. It is pointed out that, in practical operation, one can even work with dissolved oxygen concentrations other than the one used as basis for the dimensioning.

The oxygen transfer is to be determined for all relevant loading conditions. In plants without periodical fluctuations of the inflow loads during a year, the highest oxygen consumption occurs in summer. It is permitted, in summer, to work with a lower sludge age and correspondingly smaller concentrations of suspended solids in the biological reactor, and to take account of this with the calculations. If no measured results are available the calculation for T = 20° C is to be carried out. If one works in winter with a reduced denitrification volume and, as a result, higher nitrate concentrations in the effluent, verification for this is also to be undertaken. If no temperature data are available one can reckon with T = 10° C for winter conditions.

If with commissioning the loading of the plant as average of the working days is more than 30 % lower than the dimensioning loading rate, the oxygen transfer also for this is to be determined using f_N = 1 and f_C = 1 as reference value for the gradation of the aeration facility.

With large differences between the oxygen transfer of the dimensioning loading rate and the loading rate with commissioning, it can be practical first to design a smaller aeration capacity and to plan the possibility for later expansion.

With aeration facilities it is normal to tender with the oxygen transfer in clean water. The α-value for the conversion to operational conditions depends both on the type of wastewater and the properties of the activated sludge as well as on the aeration system itself. Information on this is to be taken from [1], 5.4.2.4.

Important for the economy of the operation, and also for the securing of denitrification, is the satisfactory gradation of the aeration capacity. Within a week, variation of the hourly oxygen uptake rate is at least of the ratio 7 : 1. The spread between design capacity and operational requirement, with not yet fully loaded plants, is still considerably greater, see above. The lowest oxygen consumption is to be seen at the weekend at which often, in addition, the N : BOD_5 ratio is unfavourable. With intermittent aeration there is then frequent on-off switching of the aeration device. With pre-anoxic zone denitrification, under certain circumstances, by internal recirculation a large quantity of oxygen is transferred into the denitrification tank. In both cases the degree of denitrification is reduced.

5.2.9 Alkalinity

The alkalinity (concentration of hydrogen carbonate, determination in accordance with DIN 38 409, Part 7 [available in English]) is reduced both through nitrification as well through the addition of metal salts (Fe^{2+}, Fe^{3+}, Al^{3+}) for phosphorus removal. This can also lead to a decrease in the pH value.

The alkalinity in the inflow to the biological reactor ($S_{ALK,IAT}$ in mmol/l) results primarily from the alkalinity (hardness) of the drinking water as well as the alkalinity formed by ammonification of urea and of organic nitrogen.

The alkalinity, through nitrification (with the inclusion of the recovery from denitrification) and through phosphate precipitation, decreases approximately as follows:

$$S_{ALK,EAT} = S_{ALK,IAT} - [0.07 \cdot (S_{NH4,IAT} - S_{NH4,EST} + S_{NO3,EST} - S_{NO3,IAT}) + 0.06 \cdot S_{Fe3} + 0.04 \cdot S_{Fe2} + 0.11 \cdot S_{AL3} - 0.03 \cdot X_{P,Prec}] \quad \text{[mmol/l]} \tag{5-30}$$

Here, alkalinity values are to be inserted in mmol/l and all other concentrations in mg/l. The free acid and alkali portion of certain precipitants must be taken into account separately.

The daily average remaining alkalinity is to be determined for the most unfavourable loading case, i.e., as a rule, with advanced nitrification and limited denitrification as well as for the highest precipitant dosing. If the conditions do not occur concurrently, various types of load are to be investigated.

The alkalinity should not undercut the value of $S_{ALK,EAT}$ = 1.5 mmol/l, if necessary alkaline neutralisation agents such as, for example, milk of lime, are to be added.

In deep aeration tanks (≥ 6 m) with a high oxygen transfer efficiency, despite sufficient alkalinity, the pH value can sink below 6.6 due to a too low stripping rate of the biogenous formed carbon dioxide (CO_2). Reference values can be taken from Table 9, a more accurate calculation in accordance with [1], 5.2.1.1 or [4], if required, is recommended; under certain circumstances neutralisation has to take place.

Table 9: pH values in the aeration tank dependent on the oxygen transfer efficiency and the alkalinity, calculated in accordance with [4]. The oxygen transfer efficiency is to be determined for operating conditions.

$S_{ALK,EAT}$ [mmol/l]	pH values in the aeration tank with an average oxygen transfer efficiency of				
	6%	9 %	12 %	18 %	24 %
1.0	6.6	6.4	6.3	6,1	6.0
1.5	6.8	6.6	6.5	6.3	6.2
2.0	6.9	6.7	6.6	6.4	6.3
2.5	7.0	6.8	6.7	6.5	6.4
3.0	7.1	6.9	6.8	6.6	6.5

5.3 Dimensioning of an Aerobic Selector

Aerobic selectors are practical for the reduction of the danger of filamentous bacteria growth with wastewater with a high fraction of readily biodegradable organic matter as well as in front of completely mixed aeration tanks. The reduction of BOD_5 or COD can have a negative effect on denitrification.

Anaerobic mixing tanks for excess biological phosphorus removal have a similar effect on the sludge volume index as aerobic selectors.

As guidance value for the volume of an aerobic selector a volumetric loading rate of

$B_{R,BOD}$ = 10 kg $BOD_5/(m^3 \cdot d)$ or
$B_{R,COD}$ = 20 kg $COD/(m^3 \cdot d)$ respectively

is recommended.

The oxygen transfer system should be designed for αOC = 4 kg O_2/m^3 of tank per day.

The tank should be divided at least once (2-tank cascade). Further information, in particular for concentrated wastewater from foodstuff production, is to be found in [5] and in the ATV Report "Bulking sludge, floating sludge and foam in activated sludge plants - causes and combating" [6] [available in English].

6 Dimensioning of the Secondary Settling Tank

6.1 Application Limits and Effluent Characteristics

Bases of the dimensioning are the maximum inflow rate with stormwater (Peak Wet Weather Flow rate) $Q_{WW,h}$ (m^3/h), comp. Chap. 4, the sludge volume index SVI (l/kg) and the suspended solids concentration in the influent to the secondary settling tanks SS_{EAT} (kg/m^3). With the exception of step-feed denitrification (and aeration tanks equipped with lamella separators) SS_{EAT} equals SS_{AT}.

For the design of secondary settling tanks the following are to be determined:

– shape and dimensions of the secondary settling tanks,

- permitted sludge storage and thickening time,
- return sludge flow rate as well as its control,
- type and method of operation of the sludge removal system,
- arrangement and design of the inlet and outlet.

The following dimensioning rules apply for:

- secondary settling tanks with lengths or diameters up to approximately 60 m,
- sludge volume index 50 L/kg $\leq$ SVI $\leq$ 200 L/kg,
- diluted sludge volume DSV $\leq$ 600 L/m^3,
- return sludge flow rates
  $Q_{RS} \leq 0.75 \cdot Q_{WW,h}$ (horizontal flow tanks), or
  $Q_{RS} \leq 1.0 \cdot Q_{WW,h}$ (vertical flow tanks),
- suspended solids concentration in the influent to the secondary settling tank SS_{EAT} resp. SS_{AT} > 1.0 kg/m^3.

If a further treatment stage is placed downstream, a higher concentration of settleable and/or filterable solids can be permitted in the effluent of the secondary settling tank. For this, a higher sludge volume surface loading rate and surface overflow rate are permitted. Here, the prerequisite is that the downstream stage tolerates and restrains the suspended solids occurring there.

The dimensioning of the secondary settling zone (settling pocket) of combined (aeration-settling) tanks with regard to surface overflow rate is also to be undertaken according to this Standard. With activated sludge plants with self-acting flow of return sludge, a sufficient return sludge flow rate is to be structurally ensured.

Bases for dimensioning and design are to be found in the ATV Manual [2] and in IAWQ Report No. 6 [7].

6.2 Sludge Volume Index and Permitted Thickening Time

The sludge volume index (comp Chap. 5.2.5), together with the thickening time (t_{Th}) in the secondary settling tank determines the suspended solids concentration in the bottom sludge (SS_{BS}). To prevent redissolution (of phosphate) and the formation of floating sludge as a result of unwanted denitrification in the secondary settling tank, the retention time of the settled sludge in the thickening and sludge removal zone must be kept as short as possible. On the other hand the sludge thickens better the higher the sludge layer is and the longer the retention time of the sludge in this layer is.

Due to the special significance of the thickening time t_{Th} for the dimensioning of secondary settling tanks, recommendations are made in Table 10, in dependence on the degree of wastewater treatment.

Table 10: Recommended thickening time in dependence on the degree of wastewater treatment

Type of wastewater treatment	Thickening time t_{Th} in h
Activated sludge plants without nitrification	1.5 - 2.0
Activated sludge plants with nitrification	1.0 - 1.5
Activated sludge plants with denitrification	2.0 - (2.5)

An exceeding of the thickening time of t_E = 2.0 h requires a very advanced denitrification in the biological reactor. These thickening times are achieved only with correspondingly low sludge volume index values and a small return sludge ratio.

The sludge removal system must be dimensioned that the permitted thickening time is not exceeded.

6.3 Suspended Solids Concentration in the Return Sludge

The return sludge flow rate Q_{RS} is composed by the sludge removal flow rate Q_{SR} and a short circuit sludge flow rate Q_{Short} which occurs with sludge scrapers as a flow rate from the inlet to the sludge hoppers and with suction facilities from the zone above the thickening layer. The short-circuit sludge flow rate Q_{Short} is dependent on the sludge removal flow rate and the return sludge flow rate.

The achievable suspended solids concentration in the bottom sludge SS_{BS} (average suspended solids concentration in the sludge removal flow) can be estimated empirically in dependence on the sludge volume index SVI and the thickening time t_{Th} as follows (see also Fig. 5):

$$SS_{BS} = \frac{1000}{SVI} \cdot \sqrt[3]{t_{Th}} \qquad [kg/m^3] \qquad (6\text{-}1)$$

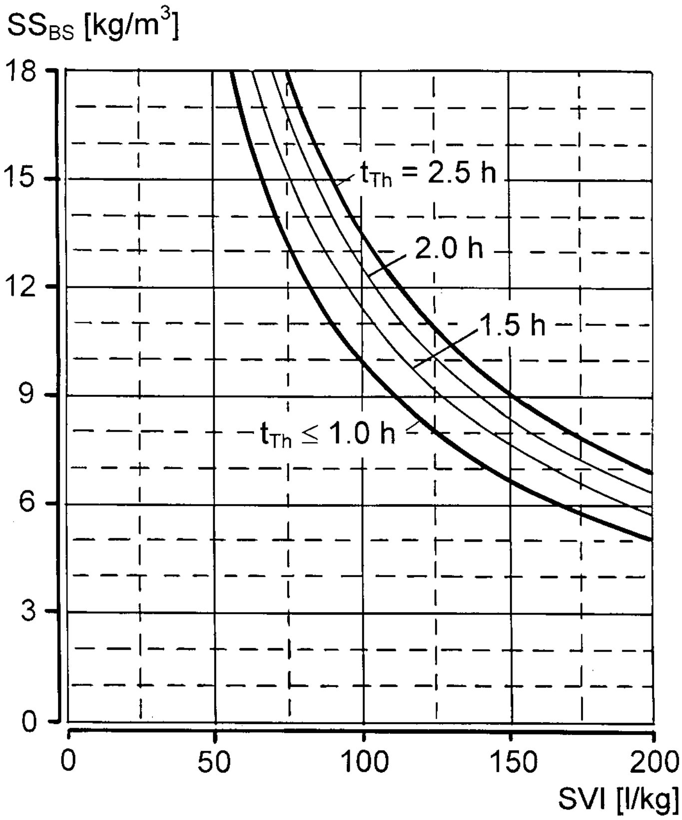

Fig. 5: Suspended solids concentration in the bottom sludge in dependence on the sludge volume index and the thickening time

A calculation of SS_{BS} using the settling velocity of the activated sludge is also possible [7].

The suspended solids concentration of the return sludge (SS_{RS}), as a result of the dilution with the short-circuit sludge flow, can be assumed in simplified form to be:

with scraper facilities $SS_{RS} \sim 0.7 \cdot SS_{BS}$

with suction facilities $SS_{RS} \sim 0.5$ to $0.7 \cdot SS_{BS}$

With vertical flow secondary settling tanks, without an installation for sludge removal $SS_{RS} \sim SS_{BS}$ can be assumed.

6.4 Return Sludge Ratio and Suspended Solids Concentration in the Influent to the Secondary Settling Tank

The operating conditions in the aeration tank and in the secondary settling tank are influenced mutually through the dependence between the mixed-liquor suspended solids concentration in the influent to the secondary settling tank SS_{EAT}, between the mixed-liquor suspended solids concentration of the return sludge SS_{RS} as well as the return sludge ratio $RS = Q_{RS}/Q$. For the equilibrium state the following results from the suspended solids mass balance, neglecting $X_{SS,EST}$:

$$SS_{AT} = \frac{RS \cdot SS_{RS}}{1 + RS} \quad [kg/m^3] \qquad (6\text{-}2)$$

The dimensioning of secondary settling tanks and aeration tanks shall be based on a maximum return sludge flow rate $Q_{RS} = 0.75 \cdot Q_{WW,h}$. The overall capacity of the return sludge pumps, i.e. including reserve, for operational reasons must be designed that the return sludge flow rate of $Q_{RS} = 1.0 \cdot Q_{WW,h}$ can be achieved. By staging of the pumps e.g. with different flow rates, it should be possible to set various return sludge ratios. A continuous matching of the return sludge flow rate to the inflow rate is, however, not necessary.

With vertical flow secondary settling tanks a maximum value of $Q_{RS} = 1.0 \cdot Q_{WW,h}$ is possible; the arrangement of the return sludge pumps (incl. reserve) should make an operational adjustment of Q_{RS} up to $1.5 \cdot Q_{WW,h}$ possible.

Return sludge ratios RS for the transition between mainly horizontal and mainly vertical flow tanks can be taken from Table 11.

Higher return sludge ratios and erratic increases of the return sludge flow prejudice the settling process through increasing flow rates. Return sludge ratios below RS = 0.5 should be avoided as they demand high suspended solids concentrations in the return sludge, which are achievable only with a low sludge volume index and a long thickening time.

6.5 Surface Overflow Rate and Sludge Volume Surface Loading Rate

The surface overflow rate q_A is calculated from the permitted sludge volume loading rate q_{SV} and the diluted sludge volume DSV as:

$$q_A = \frac{q_{SV}}{DSV} = \frac{q_{SV}}{SS_{EAT} \cdot SVI} \quad \text{[m/h]} \qquad (6\text{-}3)$$

In order to keep the concentration of suspended solids $X_{SS,EST}$ and the resulting COD and phosphorus concentration in the effluent of horizontal flow secondary settling tanks low, the following sludge volume loading rate q_{SV} shall not be exceeded:

$q_{SV} \leq 500$ l/(m^2·h) for $X_{SS,EST} \leq 20$ mg/l

For mainly vertical flow secondary settling tanks, the following applies with the formation of a closed sludge blanket or with an easily flocculating activated sludge:

$q_{SV} \leq 650$ l/(m^2·h) for $X_{SS,EST} \leq 20$ mg/l

It is recommended that an optimisation between the sludge volume loading rate and the depth of the tank is undertaken.

Predominantly horizontal flow tanks are those where the ratio of the distance from the inlet aperture to the water surface (vertical component, h_{In}) to the horizontal distance from inlet to outlet at the height of the water level (horizontal component) is smaller than 1 : 3. Predominantly vertical flow tanks are those where the ratio is higher than 1 : 2. For ratios lying between the two, the permitted sludge volume loading rate can be interpolated linearly. It is recommended that the values in Table 11 are used for dimensioning.

The surface overflow rate q_A shall not exceed 1.6 m/h with predominantly horizontal flow secondary settling tanks, and with predominantly vertical flow secondary settling tanks it shall not exceed 2.0 m/h. For the transition area values can be taken from Table 11.

Table 11: Permitted values for the transition area between predominantly horizontal and predominantly vertical flow secondary settling tanks

Ratio*)	≥ 0.33	≥ 0.36	≥ 0.39	≥ 0.42	≥ 0.44	≥ 0.47	≥ 0.5
q_{SV} (l/(m^2 · h)	≤ 500	≤ 525	≤ 550	≤ 575	≤ 600	≤ 625	≤ 650
q_A (m/h)	≤ 1.60	≤ 1.65	≤ 1.75	≤ 1.80	≤ 1.85	≤ 1.90	≤ 2.00
RS (-)	≤ 0.75	≤ 0.80	≤ 0.85	≤ 0.90	≤ 0.90	≤ 0.95	≤ 1.00

*) vertical component to horizontal component e.g. 1 : 2.5 = 0.4

6.6 Settling Tank Surface Area

The required surface area of the secondary settling tank results as follows:

$$A_{ST} = \frac{Q_{WW,h}}{q_A} \qquad [m^2] \qquad (6\text{-}4)$$

Generally only with horizontal flow secondary settling tanks additional area for the inlet disturbance zone is required. The length of this disturbance zone is, as an approximation, set equal to the side wall depth of the tank.

For vertical flow secondary settling tanks the effective surface area at the mid-point between inlet aperture and water level is to be set, see Fig. 8. With this the geometry of normal tank shapes is taken into account.

6.7 Settling Tank Depth

The various processes in secondary settling tanks are explained with the aid of functionally conditioned effective volumes, which are shown in Figs. 6 and 7.

The necessary depth of the secondary settling tank is made up from individual partial depths for the functional zones:

h_1 : clean water zone
h_2 : separation zone/return flow zone
h_3 : density flow and storage zone
h_4 : thickening and sludge removal zone

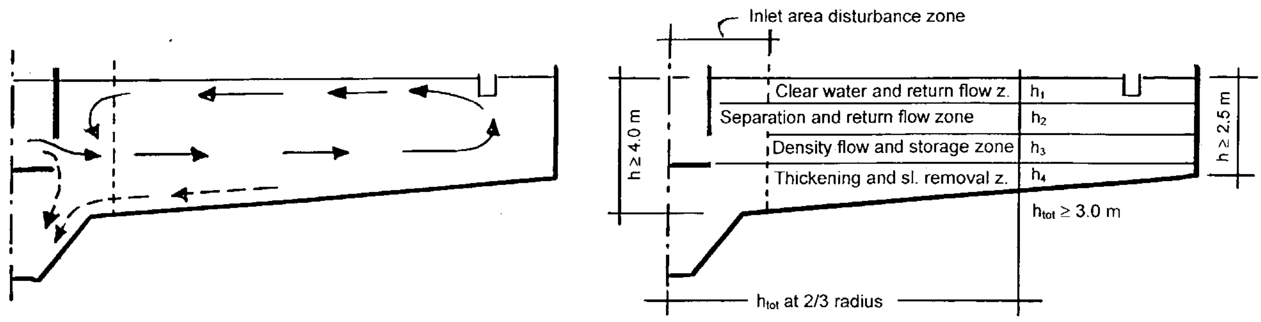

Fig. 6: Main directions of flow and functional tank zones of horizontal flow circular secondary settling tanks

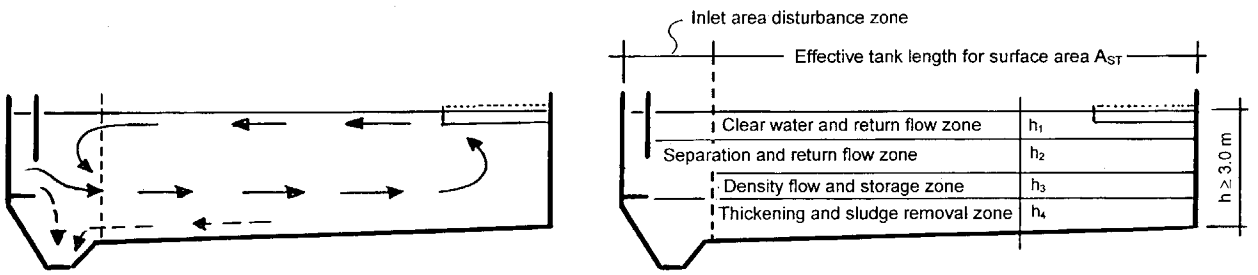

Fig. 7: Main directions of flow and functional tank zones of horizontal flow rectangular secondary settling tanks

The division into functional zones shows in which areas the process take place. In reality the processes do not take place in horizontally layered zones, but rather permeate each other. In the inlet and outlet areas of the tank there are additional hydraulically conditioned disturbance zones present, which are to be kept as small as possible through suitable inlet and outlet design.

The **clean water zone** is a safety zone with a minimum depth of **h_1 = 0.50 m**.

It serves as a means to balance out unavoidable influences from wind, differences of density or uneven surface feeding. The clean water zone often lies in the area of a return flow.

With submerged outlet pipes (tubes) a distance of 30 cm between the upper edge of the separation zone and the inlet openings of the pipes is sufficient. In order to prevent the entry of floating sludge into the outlet pipes, the water level must be at least 20 cm above the inlet openings.

Above the density flow and storage zone in the area of the inlet lies the **separation zone**. In the inlet area the separation zone and the density flow and storage zone form one unit. Here the sludge-water mixture is introduced and distributed. Flocculation processes take place which favour sludge settling. Outside the inlet area, above the density flow and storage area, lies the **return flow zone** in which, due to continuity equation, wastewater with a low suspended solids concentration flows back into the inlet area; as safety zone this includes the clean water zone.

The **separation/return flow zone** must be dimensioned that the inflow, including the return sludge flow, referred to the free water volume, has a calculated detention time of 0.5 h. From this results:

$$h_2 = \frac{0.5 \cdot q_A \cdot (1 + RS)}{1 - DSV/1000} \quad \text{[m]} \qquad (6\text{-}5)$$

The wastewater-sludge mixture, which appears in the **density flow and storage zone**, sinks down to the sludge layer due to its high density and from there flows to the outer edge of the tank. Here, the maximum velocities in the tank occur. With increasing stormwater inflow the density flow and storage zone expands. There, even with the selection of a higher return sludge ratio, the displaced sludge from the aeration tank is stored.

The **density flow and storage zone** must be dimensioned that under wet weather conditions $Q_{WW,h}$ the additional volume of sludge ($0.3 \cdot SS_{EAT} \cdot SVI$) with a concentration value of 500 l/m^3 expelled in 1.5 hours from the aeration tank can be stored. In this period the activated sludge settles in the thickening zone and is assumed to be evenly distributed over the surface area A_{ST} of the secondary settling tank.

The depth of the density flow and storage zone thus results as:

$$h_3 = \frac{1.5 \cdot 0.3 \cdot q_{SV} \cdot (1 + RS)}{500} \quad \text{[m]} \qquad (6\text{-}6)$$

The thickening of the settled sludge takes place at the bottom of the tank in the **thickening and sludge removal zone**. There a sludge layer exists in which low flow rates to the sludge hopper occur.

The thickening and sludge removal zone must be large enough so that the influent sludge load, having a suspended solids concentration of SS_{EAT}, within the thickening time t_{Th} can be thickened to the bottom sludge concentration SS_{BS}. With the assumption of an even distribution of the sludge mass over the surface of the secondary settling tank the height of the thickening and sludge removal zone results as:

$$h_4 = \frac{SS_{EAT} \cdot q_A \cdot (1 + RS) \cdot t_{Th}}{SS_{BS}} \quad \text{[m]} \qquad (6\text{-}7)$$

The calculated total depth h_{tot} of the tank for horizontal flow secondary settling tanks with inclined tank bottom is to be kept at two thirds of the flow path or radius. There it should be at least 3 m. With circular secondary settling tanks the side water depth shall not be less than 2.5 m.

With vertical flow settling tanks (see Fig. 8, inverse cone or hopper) the sub-volumes of V_2 to V_4 for the separation zone, the storage zone and the thickening zone can be calculated by multiplication of the appropriate zone depths h_2 to h_4 with the surface area A_{ST} (see Sect. 6.6).

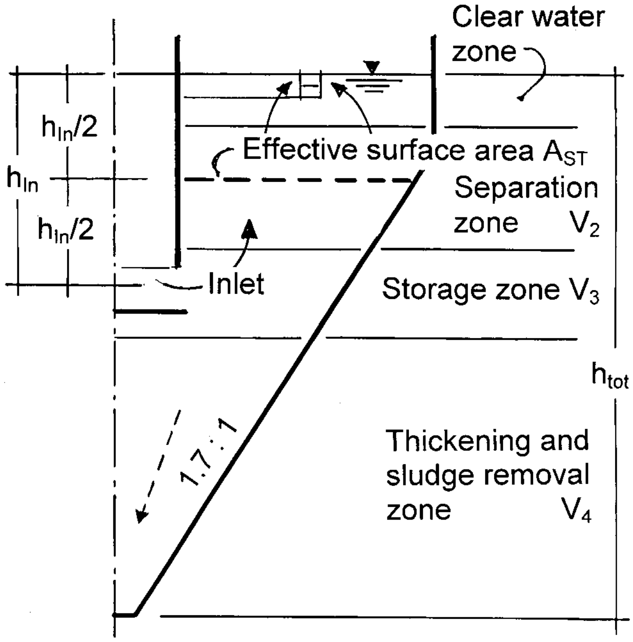

Fig. 8: Functional zones and depths of vertical flow (inverse cone) tanks

6.8 Testing and Recalculation of Existing Secondary Settling Tanks

With existing secondary settling tanks or with special local circumstances (e.g. high groundwater level), the sludge volume loading rate is to be matched to the existing or possible tank depth or is to be verified through large scale loading tests for the dimensioning case.

With the dimensioning of secondary settling tanks the permitted maximum value does not necessarily have to be selected for the sludge volume loading rate q_{SV}. With the verification of existing secondary settling tanks q_{SV} can be reduced iteratively until the calculated depth agrees with the actual depth. The tank surface area is subsequently to be verified for this sludge volume loading rate.

If the existing tank depth undercuts the required minimum value, a reduction of the maximum acceptable inflow rate is worth recommending in order to prevent hydraulic disturbances as a result of a too small tank depth. A further use of existing secondary settling tanks with a total water depth of less than 2.0 m is, in general, uneconomical and is operationally impracticable.

6.9 Design of the Sludge Removal System

6.9.1 Sludge Removal and Scraper Design

The sludge removal flow rate and the return sludge flow rate essentially determine the retention time of the activated sludge in the secondary settling tank.

For the respective types of secondary settling tanks there are various sludge scrapers and sludge return conveyance devices available. In horizontal flow circular tanks sludge scrapers and suction facilities are employed. In horizontal flow rectangular tanks, in addition to sludge scrapers and suction facilities, flight scrapers are used. If sludge removal is required in predominantly vertical flow tanks or in transverse flow settling tanks, the above-named systems can also be employed.

For dimensioning of the sludge removal devices the tank dimensions and the loads of suspended solids must have been determined.

For the arrangement of the sludge removal system attention is drawn to the ATV-DVWK Report [8] and the correction [9] as well as to the statements in [2], Chap. 3.5.4.

Guidance values for the design of sludge scrapers can be taken from Table 12.

6.9.2 Short-Circuit Sludge Flow Rate and Solids Balance

As the sludge removal flow rate Q_{SR} is often smaller than the return sludge flow rate Q_{RS}, with sludge scrapers, a short-circuit sludge flow rate Q_{Short} occurs between inlet and sludge removal point (hopper) and with suction facilities from the zone above the thickening zone. The following applies:

$$Q_{Short} = Q_{RS} - Q_{SR} \qquad [m^3/h] \qquad (6\text{-}8)$$

The short-circuit sludge flow rate Q_{Short} in accordance with Eqn. 6-8, from experience lies between 0.4 - 0.8·Q_{RS}, dependent on the return sludge flow rate.

Due to the dilution effect of the short-circuit sludge flow rate Q_{Short} the suspended solids concentration of the return sludge SS_{RS} lies below the suspended solids concentration of the bottom sludge SS_{BS} respectively in the sludge removal flow. The following solid matter balance applies:

$$Q_{RS} \cdot SS_{RS} = Q_{SR} \cdot SS_{BS} + Q_{Short} \cdot SS_{EAT} \qquad [kg/h] \qquad (6\text{-}9)$$

6.9.3 Sludge Removal in Horizontal Flow Circular Tanks

In circular tanks the removal interval is the same as the duration of one revolution of the scraper bridge :

$$t_{SR} = \frac{\pi \cdot D_{ST}}{v_{SR}} \qquad [h] \qquad (6\text{-}10)$$

The sludge removal flow rate for **sludge scrapers in circular tanks** is:

$$Q_{SR} = \frac{h_{SR} \cdot a \cdot v_{SR} \cdot D_{SR}}{4 \cdot f_{SR}} \qquad [m^3/h] \qquad (6\text{-}13^{**})$$

** Numbering of equations in agreement with original German Standard where Eqns. (6-11), (6-12), (6-16) and (6-17) are omitted.

The bridge velocity v_{SR} is referred to the tank periphery. The number of scraper arms "a" is to be selected in dependence on the tank diameter D_{SR} and the required removal flow rate.

*[**Addendum (NOT in original German text):** A scraper blade with a length of the tank radius is referred to as a = 1 number of arms.]*

For **suction facilities** (usually as organ pipe types) it is not possible to differentiate between sludge removal flow rate and short-circuit sludge flow rate because the bottom sludge generally is, in part, diluted with clarified water (mainly at the edge of the tank).

The flow rate in the suction pipes (risers) should be 0.6 to 0.8 m/s and the distance between suction pipes should not exceed 3 to 4 m. The bridge velocity v_{SR} is the same as that for scraper bridges. The suction capacity should be adjustable in stages from the middle of the tank outwards in order to keep additional hydraulic loading small.

Table 12: Guidance values for the design of sludge scrapers

	Abbr.	Unit	Circular tanks	Rectangular tanks	
			Sludge scrapers	Sludge scrapers	Flight scrapers
Scraper or beam ht.	h_{SR}	m	0.4 - 0.6	0.4 - 0.9	0.15 - 0.30
Bridge velocity	v_{SR}	m/h	72 - 144	max. 108	36 - 108
Return velocity	v_{ret}	m/h	-	max. 324	-
Removal factor*)	f_{SR}	-	1.5	≤ 1.0	≤ 1.0

*) The removal factor is the quotient of the calculated sludge removal flow rate in one removal interval and the actual sludge removal flow rate.

*[**Addendum (NOT in original German text):** The removal factor f_{SR} = 1.5 was derived from circular settling tanks with a bottom slope of 1 : 15 and spiral shaped scraper blades: 1.5 bridge revolutions are necessary to completely remove a sludge layer having a height of the scraper blade. The scraper efficiency can be visualised as the inverse of the removal factor.]*

6.9.4 Sludge Removal in Rectangular Tanks

For **sludge scrapers**, taking into account the time for lifting and lowering the scraper blade t_s (h), with the length of the runway of the scraper bridge ($L_{RW} \approx W_{ST}$) the removal interval is:

$$t_{SR} = \frac{L_{RW}}{v_{SR}} + \frac{L_{RW}}{v_{ret}} + t_s \qquad [h] \qquad (6\text{-}14)$$

In order to calculate the removal flow rate Q_{SR} the following is to be assumed: The scraper blade theoretically moves a sludge layer of a maximum length of 15 times the scraper blade height ($L_{SL} = 15 \cdot h_{SR}$). With the length of the scraper blade L_{SR} (~ b_{ST} in tanks with vertical walls) the removal flow rate becomes:

$$Q_{SR} = \frac{h_{SR} \cdot L_{SR} \cdot L_{SL}}{f_{SR} \cdot t_{SR}} \qquad [m^3/h] \qquad (6\text{-}15)$$

Convenient lengths for rectangular tanks lie below 60 m. Two rows of sludge hoppers, separated by a certain distance (less than half of the tank length), are advisable with lengths over 40 m.

Removal factors of $f_{SR} < 1.0$ for blade scrapers and flight scrapers signify that, on top of the sludge layer with the height of the blade or beam, an additional layer of sludge is transported.

For **flight scrapers**, with the length of the flight scraper ($L_{FS} \approx L_{ST}$) the removal interval is:

$$t_{SR} = \frac{L_{FS}}{v_{SR}} \quad [h] \qquad (6\text{-}18)^{**}$$

The sludge removal flow rate Q_{SR} for flight scrapers is:

$$Q_{SR} = \frac{v_{SR} \cdot L_{SR} \cdot h_{SR}}{f_{SR}} \quad [m^3/h] \qquad (6\text{-}19)$$

The distance of the beams should be approximately 15 times the beam height.

For the design of **suction facilities in rectangular tanks** the details in Sect. 6.9.3 apply. The bridge velocity should be 36 to 72 m/h. Suction facilities, depending on the momentary place of the bridge, lead to unavoidable cyclic, additional hydraulic loading in the longitudinal axis of the tank.

6.9.5 Verification of the Solids Balance

The sludge removal system must be so designed that the sludge removal flow rate Q_{SR} meets the solid matter balance in accordance with Eqn. (6-9). According to this:

$$Q_{SR} \geq \frac{Q_{RS} \cdot SS_{RS} - Q_{Short} \cdot SS_{EAT}}{SS_{BS}} \quad [m^3/h] \qquad (6\text{-}20)$$

Suspended solids concentration of the return sludge, determined under Sect. 6.3 is to be applied here for SS_{RS}.

7 Planning and Operating Aspects

7.1 Biological Reactor (Aeration Tank)

7.1.1 Tank Design

In mixing or aeration tanks with retention periods, referred to the total flow rate, of 10 minutes and less, short-circuit flows should be minimised using design measures.

In tanks with even, bottom covered arrangement of fine-bubble aeration elements, the throughflow, assumed as being homogeneous, can be partially blocked. Short-circuit flows within and by-pass flows alongside and below aeration zones can prejudice the treatment performance; circulation facilities (propellers) create inhomogeneous flow profiles, for example flow streamers. In the cases mentioned above the actual oxygen transfer rate of fine bubble diffused air systems may be smaller than as taken for dimensioning.

** Numbering of equations in agreement with original German Standard where Eqns. (6-11), (6-12), (6-16) and (6-17) are omitted.

Fundamentally basic precautions are to be taken which allow repairs of equipment located in the tank while maintaining correct operation. Channels and pump sumps for the collection of the activated sludge are useful for the emptying of tanks.

7.1.2 Accumulation of Foam and Floating Sludge

Foam and floating sludge can develop, in particular with the occurrence of Microthrix parvicella, on aeration tanks, however also on denitrification tanks and, with certain conditions, also on anaerobic mixing tanks. In order to minimise an accumulation of frothy sludge, the partition walls in tanks should always be overflowed. During filling and emptying, a high one-sided water pressure on the separation walls is avoided through a small opening near the bottom. For the same reason scum baffles in front of effluent channels of aeration tanks are not practical. Often the foam in the channel is broken up by the overflowing water.

As the occurrence of Michrothrix Parvicella has not up until now been mastered, a possibility for the removal of the scum should be planned as a preventative measure. This can, for example, be at the distributor to the secondary settling tanks or at a common, open effluent channel of the aeration tanks. Here it must be possible to install a suitable suction device. The removed foam should, as far as possible, not be fed to the digester without further treatment; it can, for example, be applied to drying beds.

7.1.3 Regulation of the Pumps for Internal Recirculation

Due to the small difference in water head of the internal recirculation system, in many cases the pump flow rate is to be determined approximately only. In order to prevent a too high recirculation and thus a high oxygen transfer to the denitrification zone, a throttling or, better, remote control is sensible.

7.1.4 Nitrite Formation in Plants not Dimensioned for Nitrification

Under certain conditions (high temperature, low loading) nitrification can occur occasionally in plants, which are designed solely for carbon removal. With this the oxygen consumption increases and one has to reckon with increased nitrite concentrations in the effluent. These adverse effects are to be countered through a greater oxygen transfer or, if this is not possible, through reduction of the sludge age (increase of the waste sludge flow rate).

7.2 Secondary Settling Tanks

7.2.1 General

In this STANDARD only design aspects are dealt with which affect dimensioning or which have been assumed with the dimensioning. Further planning aspects of design and arrangement, for example due to site or underground constraints, the process of construction, traffic safety or similar are not listed here explicitly. For this see the ATV Handbook [2], Chap. 3.5 and the ATV Report [10].

7.2.2 Mainly Horizontal flow Tanks

Tank size

Usual tank diameters for circular tanks lie at about 30 to 50 m. The even removal of clean water from large circular tanks with peripheral launders can be disturbed by the influence of wind. Circular tanks with diameters less than 20 m should be calculated and designed as vertical flow tanks (Chaps. 6.5 and 6.7).

Inlets

The design of the inlet influences the separation performance in secondary settling tanks substantially.

The inflowing mixture of wastewater and activated sludge must be distributed as evenly as possible in the inlet area and discharged horizontally into the separation zone or into the density flow and storage zone respectively. With lower lying inlet apertures attention is to be paid to short-circuit flows.

Volumes for flocculation and gas stripping are useful before entry into the settling zone - in particular with deep aeration tanks. Stripping of gas can be performed using a presited degasing area in the inlet zone or in distribution channels or in the last part of the aeration tank. A removal of floating sludge should be considered to take place here. A flocculation should be supported through a moderate flow rate in the inlet area of up to 40 cm/s over a period of 3 to 5 minutes until entry into the tank. For the idealised horizontal component of the entry velocity into circular tanks a calculated value of up to 10 cm/s is advisable. With rectangular tanks the horizontal component of the flow velocity may not exceed 0.25 to 1.33 cm/s, due to the limitation of the surface loading rate. The inlet volumes of circular tanks and distribution flumes of rectangular tanks should be dimensioned for a detention time of 1 minute for peak wet weather flow including return sludge ($Q_{WW,h} \cdot (1+RV)$).

Outlets

The separation of wastewater and sludge carried out in the tank must be ensured through a hydraulically satisfactory outlet design. Therefore inboard launders must be fixed with a sufficient distance to the side wall. The launder overflow rate must be limited to 10 $m^3/(m \cdot h)$; with launders fed on both sides, to 6 $m^3/(m\ h)$. With sludge volume indices above 150 l/kg, these values are to be reduced further.

A more uniform withdrawal from the effluent surface area supports the requirement for an undisturbed removal of the clean water. This may be achieved with radially arranged, submerged, perforated outlet tubes [11] or several launders. Possible variations in water level must be taken into account. A scumboard must be arranged in a distance of 30 cm from the launder in order to avoid the overflow of floating sludge. The submerged depth should be 20 cm.

Sludge hoppers

Secondary settling tanks with sludge scrapers do not require large sludge hoppers if no additional thickening is required within them. The hoppers must be designed that no depositing of sludge is to be expected. The hopper walls should be as smooth as possible and have a slope of at least 1.7 : 1. For longitudinal tanks with inverse pyramid hoppers attention is to be paid to a rounding off of the crests.

7.2.3 Mainly Vertical flow Tanks

Vertical flow secondary settling tanks can be constructed as circular or rectangular tanks. Normally they are deeper than horizontal flow secondary settling tanks. The ratio of the vertical component h_{In} to the horizontal component up to the outer tank wall at the height of the water level should, as far as possible, be higher than 1 : 2 in order that a sludge blanket can be formed.

Circular and hopper-bottom tanks

The hopper shaped tank (Dortmund tank) is the most frequent form of construction of vertical flow secondary settling tanks. The hopper (conical) shape provides an even upwards flow distribution and encourages the formation and stability of a suspended sludge blanket. At least 75 % of the tank depth should be shaped in a hopper form. A hopper slope of 1.7 : 1 is practical. Flatter slopes down to 1.4 : 1 are only possible with very smooth and particularly carefully produced wall surfaces. Usually the hopper slope is also continued into the thickening volume, so that no sludge removal system is required.

With circular tanks with flat bottoms, a scraper must transport the sludge to the removal point.

Rectangular tanks

Vertical flow rectangular tanks are usually constructed as longitudinal tanks with flat bottoms. They are flown transversely, whereby the even inlet distribution over the complete tank length takes on a special significance. The sludge removal takes place preferably through a bridge mounted organ pipe suction facility, which is moved longitudinally or, with small tanks up to 25 m in length, using a suction pipe on a trough-shaped tank bottom.

Inlet

The inlet design for vertical flow rectangular and circular tanks with flat bottoms equals that of horizontal flow secondary settling tanks.

In hopper shaped tanks the inlet is a central construction with a submerged cylinder and a flow diversion on entry into the tank. The lower side of the submerged cylinder should end above the thickening zone, advisable is the centre of the storage zone. The diameter of the inlet cylinder should be selected as 1/5 to 1/6 of the diameter of the relevant dimensioning surface area.

In transverse flow rectangular tanks the inflow must be deep and evenly distributed into the tank.

Outlet

The design of the outlet with vertical flow secondary settling tanks can be constructed similar to horizontal flow tanks.

In circular tanks and in hopper-bottom tanks a radial arrangement of launders or radial outlet tubes aids the even flow in the secondary settling space. With this, submerged outlet tubes have the advantage that a removal of floating sludge is not obstructed. Planar draw-off of the clean water favours the hydraulic efficiency with rectangular tanks also. In rectangular tanks along both sidewalls effluent launders should be arranged.

7.3 Return Sludge

The control or regulation of the return sludge flow rate has a great significance. Objectives of the operating strategy should be:

– ensuring the necessary return of the activated sludge for the maintenance of the desired mixed liquor suspended solids concentration in the aeration tank,

– closing of the sludge circulation loop between settling-, thickening-, sludge removal process and aeration tank,

– if required, support of the equalization of the hydraulic feeding of the secondary settling tank and maintenance of a sludge blanket.

With a continuous or quasi-continuous automatic control of the return sludge flow rate to the influent flow rate (RS constant), a constant flow ratio of ca. 0.75 to 1 times the dry weather inflow $Q_{DW,h}$ should be maintained. To avoid a high hydraulic loading with the start-up of stormwater influent an increasing of the return sludge flow rate should be time delayed and should start up smoothly. For example it can be controlled according to a sliding 1 to 2 hour average of the inflow rate.

Acquisition and recording of the return sludge flow rate is necessary; in addition the recording of the sludge blanket height in at least one of the secondary settling tanks is desirable.

8 Dynamic Simulation

A new step has been taken with the description of the processes in activated sludge plants by taking into account system and process knowledge through dynamic modelling. The employment of such models was initially limited to the university level. After the publication of the "Activated Sludge Model No. 1" [12] and its conversion into suitable PC programmes, dynamic simulation has gained in significance.

Dynamic simulation today is already frequently used to check the operating behaviour of statically dimensioned activated sludge plants. Here, the process configuration and the measurement and control technology are varied and optimisation elaborated.

With simple (one dimensional) secondary settling tank models, which are coupled with an activated sludge model the sludge relocation between biological reactor and secondary settling tank can be recorded dynamically and, with this, the informative value of the modelling of the activated sludge process improved. With the aid of hydrodynamic (2 and 3 dimensional) models the function of pre-dimensioned secondary settling tanks can be examined and the design of the structure can be optimised for flow technology. Areas and limits of application of different types of model are discussed in an ATV Report [14].

Which special tasks can be elaborated with the aid of simulation depends significantly on the basic models. A model can describe sufficiently only those questions which are taken into account with the formation of the model. For the inexperienced user there is thus a danger that he considers only very simply the question to be processed. The fact that the dimensioning as a rule takes place for only one type of load, leads to the situation where the simulation is also based on only a single type of load.

It lies in the nature of simulation that uncertainties and bottlenecks are not forecast by the modelling, but rather that, through suitable assumptions (operating concepts, types of load and considerations of sensitivity) of various situations have to be made in advance, assessed and included in considerations. For this comp. The ATV Report [13].

Therefore, high requirements have to be placed on the users of simulation models which refer not only to model knowledge, but rather also the selection of type of load and the characteristics of the process.

Under these prerequisites the arrangement of activated sludge plants can be optimised with regard to safety and economic efficiency using dynamic simulation.

9 Costs and Environmental Effects

The Standard, in comparison with the previous version, is based on dimensioning and operating experience which has, in the meantime, been secured. In this respect some specifications previously introduced only as estimates or assumptions can be replaced by appropriately clear statements and the designation of influencing parameters - and in many cases, also confirmed.

With this Standard, planners and examiners receive a differentiated working basis for dimensioning single-stage activated sludge plants. From this they can, from the process technical aspect, develop the most suitable and economic solution with regard to the necessary environmental protection. Here, the possibility of variant and sensitivity investigations and through these an improved integration into the overall planning process are to be emphasised.

The requirements on the quality of the wastewater to be discharged into surface waters are not established in this Standard; they are either laid down legally or are determined in the conversion of legal specifications. This Standard is aimed at the secure maintenance of the appropriate specifications and an economic operation.

10 Relevant [German] regulations, directives and standard specifications

[Translator's note: known translations are given in English, otherwise a courtesy translation is provided in square brackets]

- **Abwasserverordnung [Wastewater Ordinance]**

Verordnung über Anforderungen an das Einleiten von Abwasser in Gewässer (AbwV) [Ordinance on the Requirements on the Discharge of Wastewater into Surface Waters] Bundesgesetzblatt 1999, Part 1, No. 6 dated 18.02.99

- **ATV Set of Rules and Standards**

ATV-A 122E
Principles for Dimensioning, Construction and Operation of Small Sewage Treatment Plants with Aerobic Biological Purification Stage for Connection Values between 50 and 500 Total Number of Inhabitants and Population Equivalents, Issue 06/91

ATV-A 126E
Principles for Wastewater Treatment in Sewage Treatment Plants according to the Activated Sludge Process with Joint Sludge Stabilisation with Connection Values between 500 and 5,000 Total Number of Inhabitants and Population Equivalents, Issue 12/93

ATV-A 128E
Standards for the Dimensioning and Design of Stormwater Structures in Combined Sewers, Issue 04/92

ATV-A 202
Verfahren zur Elimination von Phosphor aus Abwasser
[Methods for the elimination of phosphorus from wastewater]
Issue 10/92

ATV-M 209E
Measurement of the Oxygen Transfer in Activated Sludge Aeration Tanks with Clean water and in Mixed Liquor, Issue 06/96

ATV-M 210
Belebungsanlagen mit Aufstaubetrieb
[Activated sludge plants with impoundage operation]
Issue 09/97

ATV-M 256
Steuern und Regeln der N-Elimination beim Belebungsverfahren
[Control and regulation of N elimination with activated sludge processes]
Issue 1997

ATV-M 265
Regulung der Sauerstoffzufuhr beim Belebungsverfahren
[Regulation of the oxygen transfer with activated sludge processes]
Issue 2000

ATV-M 271
Personalbedarf für den Betrieb kommunaler Kläranlagen
[Personnel requirement for the operation of municipal sewage treatment plants]
Issue 09/98

- **Standard Specifications**

DIN EN 1085

Wastewater treatment - Vocabulary; Trilingual version EN 1085:1997

DIN 4045

Wastewater engineering - Vocabulary

DIN 4261, Part 2
Small sewage treatment plants; plants with sewage aeration; application, design, construction and testing

DIN 18202

Dimensional tolerances in building construction; Buildings

DIN 19558

Überfallwehr mit Tauchwand, getauchte Ablaufrohre in Becken; Baugrundsätze, Hauptmaße, Anwendungsbeispiele
[Overfall weir with scum board, submerged outlet pipes in tanks; construction principles, main dimensions, examples of application]

DIN19569-1

Principles for the design of structures and technical equipment for sewage treatment plants; General principles

DIN19569-2

Principles for the design of structures and technical equipment for sewage treatment plants; Installations for separating and thickening solids

Draft DIN EN 12255-1

Wastewater treatment plants; Part 1: General construction principles

Draft DIN EN 12255-4

Wastewater treatment plants; Part 4: Primary settling stage

Draft DIN EN 12255-6

Wastewater treatment plants; Part 6: Activated sludge process

Draft DIN EN 12255-8

Wastewater treatment plants; Part 8: Sludge treatment and storage

Draft DIN EN 12255-10

Wastewater treatment plants; Part 10: Safety principles

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2. ATV (publisher): ATV-Handbuch „Mechanische Abwasserreinigung“ [ATV Manual “Mechanical wastewater treatment”]. 4th Issue,: Berlin: Ernst & Sohn, 1997.

3. ATV-Arbeitsblatt „Vereinheitlichung und Herleitung von Bemessungswerten für Abwasseranlagen“ [ATV Standard “Unification and derivation of dimensioning values for wastewater systems”] (in preparation).

4. Nowak, O.: Nitrifikation im Belebungsverfahren bei maßgebendem Industrieabwassereinfluss [Nitrification in activated sludge processes with relevant industrial wastewater influent] Wiener Mitteilungen Wasser, Abwasser, Gewässer, Vol. 135 (1996).

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8. ATV Report "Schlammräumsysteme für Nachklärbecken von Belebungs-anlagen" [Sludge clearance systems for secondary settling tanks in activated sludge plants] Korrespondenz Abwasser 35 (1988), 263 ff.

9. Correction: Corrections to the ATV Report [8]. Korrespondenz Abwasser 35 (1988), 611.

10. ATV Report "Konstruktive Aspekte der Planung von Nachklärbecken von Belebungsanlagen" [Design aspects of the planning of secondary settling tanks in activated sludge plants]. Korrespondenz Abwasser 44 (1997), 2061-2064 and 45 (1998), 549.

11. ATV Report "Bemessung und Gestaltung getauchter, gelochter Ablaufrohre in Nachklärbecken" [Dimensioning and design of submerged, perforated discharge pipes in secondary settling tanks]. Korrespondenz Abwasser 42 (1995), 1851-1852 and 44 (1997), 322-324.

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13. ATV Report "Simulation von Kläranlagen" [Simulation of sewage treatment plants]. Korrespondenz Abwasser 44 (1997), 2064-2074.

14. ATV Report "Grundlagen und Ersatzbereich der numerischen Modellierung der Nachklärbecken von Belebungsanlagen" [Bases and scope of numerical modelling of secondary settling tanks in activated sludge plants]. Korrespondenz Abwasser 47 (2000).

Appendix

Determination of the sludge production and the oxygen consumption for carbon removal on the basis of the COD

A1 Dimensioning Principles

For the calculations, the relevant loads or the concentrations and the associated daily inflow to the biological reactor are required of:

$C_{COD,IAT}$	Chemical oxygen demand (COD)
$S_{COD,IAT}$	COD of the filtrate (0.45 μm membrane filter)
$X_{COD,IAT}$	COD of the filterable solids
$X_{SS,IAT}$	Filterable solids (0.45 μm membrane filter)
$X_{inorgSS,IAT}$	Inorganic residue of the filterable solids ($X_{SS,IAT}$)

The relevant loads are to be determined in accordance with Chap. 4.

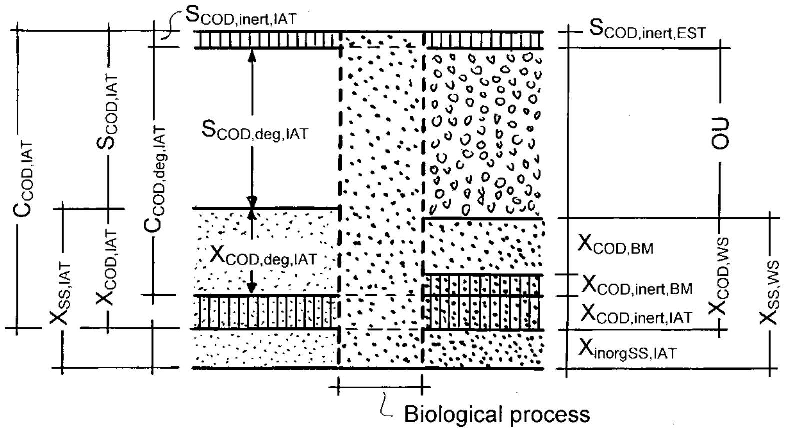

Fig. A1: Change of COD and the filterable solids with biological treatment (diagram of principle)

A2 COD Balance

The COD in the inflow to a biological reactor can be divided into the soluble and the particulate fraction. Here, attention is to be paid that all concentrations are related to the wastewater inflow; this also applies for OV, $X_{COD,SP}$ etc, comp. Fig. A1.

$$C_{COD,IAT} = S_{COD,IAT} + X_{COD,IAT} \quad \text{[mg/l]} \tag{A1}$$

Each of the two fractions consist of a biodegradable and an inert fraction:

$$C_{COD,IAT} = S_{COD,deg,IAT} + S_{COD,inert,IAT} + X_{COD,deg,IAT} + X_{COD,inert,IAT} \quad \text{[mg/l]} \tag{A2}$$

The soluble inert fraction can be approximated to the soluble effluent concentration:

$$S_{COD,inert,IAT} = S_{COD,inert,EST} \quad [mg/l] \tag{A3}$$

The inert soluble COD lies between 0.05 and 0.1 $C_{COD,IAT}$. If no readings are available it is recommended for municipal wastewater to calculate $S_{COD,inert,IAT} = 0.05 \cdot C_{COD,IAT}$. The inert part of the particulate COD can also be estimated as part of the total particulate COD:

$$X_{COD,inert,IAT} = A \cdot X_{COD,IAT} = A \cdot (C_{COD,IAT} - S_{COD,IAT}) \quad [mg/l] \tag{A4}$$

Depending on the type of wastewater and retention time in the primary settling tank, A can lie between 0.2 and 0.35. It is recommended to reckon with A = 0.25 for municipal wastewater.

The biodegradable COD ($C_{COD,deg,IAT}$) can be given as follows:

$$C_{COD,deg,IAT} = C_{COD,IAT} - S_{COD,inert,EST} - X_{COD,inert,IAT} \quad [mg/l] \tag{A5}$$

If external carbon is added regularly to improve denitrification, $S_{COD,deg,IAT}$ is to be increased by the value of $S_{COD,Ext}$ (see Eqn. 5-8). $S_{COD,Ext} \leq 10$ mg/l is not taken into account.

The filterable solids of the inflow ($X_{SS,IAT}$) consist of organic and inorganic fractions; the latter is not part of $C_{COD,IAT}$.

$$X_{SS,IAT} = X_{orgSS,IAT} + X_{inorgSS,IAT} \quad [mg/l]$$

or:

$$X_{inorg,SS,IAT} = B \cdot X_{SS,IAT} \quad [mg/l] \tag{A6}$$

The value of B can be set as 0.2 to 0.3 (70 % to 80 % organic). If no readings are available it is recommended that, for raw wastewater one reckons with B = 0.3, and for effluent from primary settling with B = 0.2.

After many measurements the organic dry solid matter in the inflow has 1.45 g COD/g org SS. With this the following correlation can be made:

$$X_{COD,IAT} = C_{COD,IAT} - S_{COD,IAT} = X_{SS,IAT} \cdot 1.45 \cdot (1 - B) \quad [mg/l] \tag{A7}$$

If $S_{COD,IAT}$ is unknown, but $X_{SS,IAT}$ has been measured, one can estimate $S_{COD,IAT}$ from this equation.

As a result of the biological treatment the COD in the effluent from the secondary settling tank (which is made up from the soluble inert COD, the non-biodegraded soluble COD and the COD from filterable solids) and the waste activated sludge measured as COD ($X_{COD,SP}$) are remaining. The difference is represented by the oxygen used for respiration (OU). If one ignores the non-biodegraded, soluble biodegradable COD in the effluent and considers the suspended solids in the effluent as misdirected waste sludge, the following conservation equation is obtained:

$$C_{COD,IAT} = S_{COD,inert,EST} + X_{COD,SP} + OV \quad [mg/l] \tag{A8}$$

In view of the high sludge age one can assume complete conversion of both the biodegradable particulate substances ($X_{COD,deg,IAT}$) as well as of the biodegradable soluble substances

($S_{COD,deg,IAT}$). The slight increase of the inert soluble COD as well as the inorganic solid matter as a result of the biodegradation process are ignored for further considerations.

A3 Calculation of the Sludge Production

The sludge produced, measured as COD, ($X_{COD,SP}$) is made up from the inert particulate influent COD, the biomass formed ($X_{COD,BM}$) and the inert solid matter ($X_{COD,inert,BM}$) remaining from the endogenous decay of the biomass.

$$X_{COD,SP} = X_{COD,inert,IAT} + X_{COD,BM} + X_{COD,inert,BM} \quad \text{[mg/l]} \tag{A9}$$

For the formation and the endogenous decay of biomass the following correlation applies:

$$X_{COD,BM} = C_{COD,deg,IAT} \cdot Y - X_{COD,BM} \cdot t_{SS} \cdot b \cdot F_T \quad \text{[mg/l]} \tag{A10}$$

$$X_{COD,BM} = C_{COD,deg,IAT} \cdot Y \cdot \frac{1}{1 + b \cdot t_{SS} \cdot F_T} \quad \text{[mg/l]} \tag{A11}$$

$$F_T = 1.072^{(T-15)} \tag{A12}$$

The yield factor Y = 0.67 g COD/g COD_{deg} and the decay coefficient b = $0.17d^{-1}$ at 15° C are both assumed analogous to those in Activated Sludge Model No. 1 [12].

The inert solid matter remaining from the endogenous decay can be set as 20 % of the decayed biomass:

$$X_{COD,inert,BM} = 0.2 \cdot X_{COD,BM} \cdot t_{SS} \cdot b \cdot F_T \quad \text{[mg/l]} \tag{A13}$$

The mass of solid matter, which is recorded as COD ($X_{COD,SP}$) is 80 % organic. If one reckons with 1.45 g COD/g SS and taking into account the inorganic filterable substances of the influent, one the obtains:

$$SP_{d,C} = Q_d \cdot \left(\frac{X_{COD,SP}}{0.8 \cdot 1.45} + X_{inorgSS,IAT}\right)/1000 \quad \text{[kg SS/d]} \tag{A14}$$

or:

$$SP_{d,C} = Q_d \cdot \left(\frac{X_{COD,SP}}{0.8 \cdot 1.45} + B \cdot X_{SS,IAT}\right)/1000 \quad \text{[kg SS/d]} \tag{A15}$$

A4 Calculation of the Oxygen Uptake

Oxygen uptake is derived from rearranged Eqn. A 8:

$$OU = C_{COD,IAT} - S_{COD,inert,EST} - X_{COD,SP} \quad \text{[mg/l]}$$

$$OU_{d,C} = Q_d \cdot (C_{COD,IAT} - S_{COD,inert,EST} - X_{COD,SP})/1000 \quad \text{[kg } O_2\text{/d]} \tag{A16}$$

Further calculations are to be carried out in accordance with Sect. 5.2.8.