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BSI Standards Publication

Wastewater treatment plants

Part 6: Activated sludge process

bsi.

National foreword

This British Standard is the UK implementation of EN 12255-6:2023. It supersedes BS EN 12255-6:2002, which is withdrawn.

The UK participation in its preparation was entrusted to Technical Committee B/505/40, Wastewater Treatment Plants > 50 PT.

A list of organizations represented on this committee can be obtained on request to its committee manager.

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Contents

European foreword

This document (EN 12255-6:2023) has been prepared by Technical Committee CEN/TC 165 “Waste water Engineering”, the secretariat of which is held by DIN.

This European Standard shall be given the status of a national standard, either by publication of an identical text or by endorsement, at the latest by January 2024, and conflicting national standards shall be withdrawn at the latest by January 2024.

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. CEN shall not be held responsible for identifying any or all such patent rights.

This document supersedes EN 12255-6:2002.

This is the sixth part prepared by Working Group CEN/TC 165/WG 40, relating to the general requirements and processes for treatment plants for a total number of inhabitants and population equivalents (PT) over 50.

The EN 12255 series with the generic title “Wastewater treatment plants” consists of the following Parts:

- *Part 1: General construction principles*
- *Part 2: Storm management systems*
- *Part 3: Preliminary treatment*
- *Part 4: Primary treatment*
- *Part 5: Lagooning processes*
- *Part 6: Activated sludge process*
- *Part 7: Biological fixed-film reactors*
- *Part 8: Sludge treatment and storage*
- *Part 9: Odour control and ventilation*
- *Part 10: Safety principles*
- *Part 11: General data required*
- *Part 12: Control and automation*
- *Part 13: Chemical treatment — Treatment of wastewater by precipitation/flocculation*
- *Part 14: Disinfection*
- *Part 15: Measurement of the oxygen transfer in clean water in aeration tanks of activated sludge plants*
- *Part 16: Physical (mechanical) filtration*

NOTE Part 2 is under preparation.

NOTE For requirements on pumping installations at wastewater treatment plants see EN 752, *Drain and sewer systems outside buildings — Sewer system management* and EN 16932 (all parts), *Drain and sewer systems outside buildings — Pumping systems*.

Any feedback and questions on this document should be directed to the users' national standards body. A complete listing of these bodies can be found on the CEN website.

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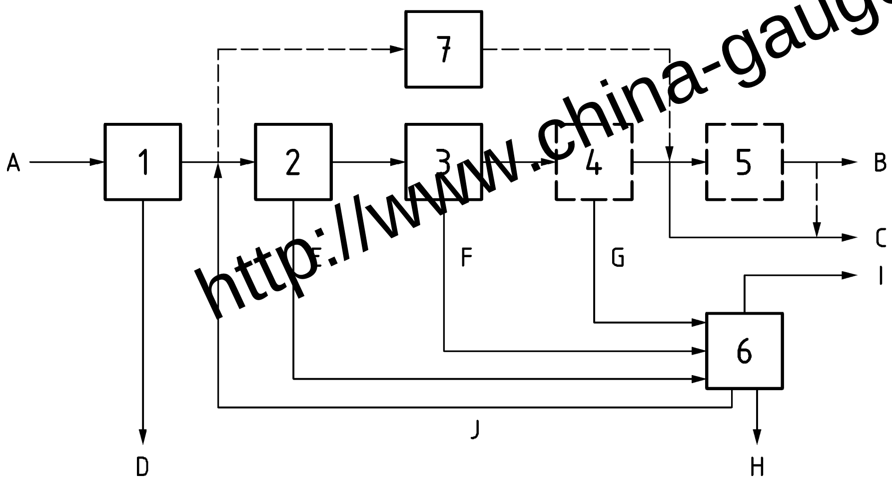
Introduction

Differences in wastewater treatment throughout Europe have led to a variety of systems being developed. This document gives fundamental information about the systems; this document has not attempted to specify all available systems. A generic arrangement of wastewater treatment plants is illustrated in Figure 1:



Key:

1 preliminary treatment
2 primary treatment
3 secondary treatment
4 tertiary treatment
5 additional treatment (e.g. disinfection or removal of micropollutants)
6 sludge treatment
7 lagoons (as an alternative)
A raw wastewater
B effluent for re-use (e.g. irrigation)
C discharged effluent
D screenings and grit
E primary sludge
F secondary sludge
G tertiary sludge
H digested sludge
I digester gas
J returned water from dewatering

Figure 1 — Schematic diagram of wastewater treatment plants

The primary application is for wastewater treatment plants designed for the treatment of domestic and municipal wastewater.

NOTE For requirements on pumping installations at wastewater treatment plants see EN 752, *Drain and sewer systems outside buildings*, and EN 16932, *Drain and sewer systems outside buildings — Pumping systems*:

— *Part 1: General requirements;*

— *Part 2: Positive pressure systems;*

— *Part 3: Vacuum systems.*

1 Scope

This document specifies performance requirements for treatment of wastewater using the activated sludge process for plants over 50 PT.

The informative Annexes A to W provide design information.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

EN 16323, *Glossary of wastewater engineering terms*

EN 12255-1, *Wastewater treatment plants - Part 1: General construction principles*

EN 12255-10, *Wastewater treatment plants - Part 10: Safety principles*

EN 12255-11, *Wastewater treatment plants - Part 11: General data required*

EN 12255-12, *Wastewater treatment plants - Part 12: Control and automation*

3 Terms and definitions

For the purposes of this document, the terms and definitions given in EN 16323 and the following apply.

ISO and IEC maintain terminology databases for use in standardization at the following addresses:

- ISO Online browsing platform: available at https://www.iso.org/obp/ui
- IEC Electropedia: available at https://www.electropedia.org/

3.1
enhanced biological phosphorus removal
activated sludge system for increased biological phosphorus removal by luxury uptake whereby mixed liquor or return sludge is intermittently subjected to anaerobic and aerobic conditions

3.2
internal recirculation ratio
IRR
ratio of the flow of recirculated nitrate containing wastewater to a denitrification reactor relative to the inflow

3.3
selector
first, optional reactor of an activated sludge system where incoming wastewater and return activated sludge are blended and mixed to subject the return activated sludge to a high sludge load in order to mitigate sludge bulking

Note 1 to entry: A selector can be aerobic or anaerobic; aerobic selectors are more common. An anaerobic selector can also be used to assist biological phosphorus removal.

3.4
mixed liquor suspended solids
MLSS
dry mass concentration of suspended solids in a mixed liquor

[SOURCE: EN 16323:2014, definition 2.3.10.24]

Note 1 to entry: The dry mass of filtered solids is determined in accordance with the 23rd edition of Standard Methods for Wastewater (SMEWW), 2540 parts D & E.

3.5
mixed liquor volatile suspended solids
MLVSS
dry mass concentration of organic suspended solids in a mixed liquor

[SOURCE: EN 16323:2014, definition 2.3.10.25]

Note 1 to entry: The dry mass of filtered solids is determined in accordance with the 23rd edition of Standard Methods for Wastewater (SMEWW), 2540 parts D & E.

4 Symbols and abbreviations

4.1 Symbols

Symbol	Definition	Unit
A	area	m^2
C	mass concentration	mg/l
D	diameter	m
OUR_{spec}	oxygen uptake rate per person (specific oxygen consumption)	kg/(P·d)
f_U	utilization factor (see EN 12255-1)	(dimensionless)
F/M	load (food to mass ratio), (e.g. kg (BOD_5/d) per kg MLSS)	kg/(kg·d)
HRT	hydraulic retention time (= V/Q)	d or h
IRR	internal recirculation ratio (for recirculation of nitrate)	(dimensionless)
L	length	m
$MASRT$	aerobic sludge age = mean aerobic solids retention time	d
$MSRT$	sludge age = mean solids retention time	d
OC	oxygen (transfer) capacity	kg/h
OC_{spec}	specific oxygen consumption per person	kg/(P· d)
OTE	oxygen transfer efficiency at operational conditions	kg/kWh
P	power	W or kW

Symbol	Definition	Unit
PT	total population (= population + population equivalents)	P
Q	flow	m^3/h or l/s
Q_{spec}	specific flow per person	m^3/(P·h)
RSR	return sludge ratio = return sludge flow to wastewater inflow	(dimensionless)
SOTR	standard oxygen transfer rate in clean test water	kg/h
SSOTR	specific standard oxygen transfer rate in clean test water per standard volume of air	g/(Nm^3·h)
SOTE	standard oxygen transfer efficiency in clean test water	kg/kWh
SSOTE	specific standard oxygen transfer efficiency in clean test water (percent of supplied oxygen transferred per immersion depth)	%/m
SSP	surplus sludge production	kg/d
SSP_{spec}	specific surplus sludge production per person	g/(P·d)
SVI	sludge volume index	ml/g
SSVI	stirred sludge volume index	ml/g
T	temperature	°C or K
V	volume	m^3
W	width	m
Y	yield (generated biomass per mass of substrate)	kg/kg
a_S	number of scraper arms	-
b	degradation rate	d^{-1}
c	molar concentration	mol/m^3
f	factor	(dimensionless)
h	height or depth	m
l	load per person and day	g/(P·d)
m	mass	g or kg
n	number of scraper arms or diffusers	—
p	pressure	Pa, hPa or kPa
q	specific flow relative to x	m^3/(h·x)
t	time	d, h or s
v	velocity	m/s

Symbol	Definition	Unit
α	alpha factor = ratio of oxygen transfer coefficients in wastewater to clean test water	(dimensionless)
β	salinity factor of clean test water	(dimensionless)
Δp	pressure loss	Pa or kPa

4.2 Indices (not included in the symbols or abbreviations below)

Al3	trivalent aluminium (Al^{3+})
aer	aeration
alk	alkalinity
atm	atmospheric (ambient)
B	bottom
BioP	enhanced biological P removal
Bl	blower
BM	biomass
Cla	clarifier
cy	cycle
Deg	degraded or degradable
del	delay (times for raising and lowering a scraper blade)
Den	denitrification
des	design
Dif	diffuser
dis	dissolved
Dos	dosing
DS	dried solids
eff	effective
Fe	iron
Fe2	bivalent iron (Fe^{2+})
Fe3	trivalent iron (Fe^{3+})
geo	geodetic (vertical level)
h	hourly
im	immersion
in	incoming
inert	not degradable
inorg	inorganic
int	intermittent aeration

intD	intermittent denitrification
kLA	oxygen transfer coefficient
max	maximal
min	minimal
Nitr	nitrification
org	organic
out	outgoing
part	particulate
PL	Pipeline
PostD	post-denitrification
PreD	pre-denitrification
prec	precipitated
Proc	process
R	reactor
redeg	readily degradable
ret	returned
S	scraper
Sal	salinity
Sat	saturation
SC	shortcut
Scr	scraper
SE	scraper effectiveness
SimD	simultaneous denitrification
Spec	specific (related to x)
St	standard
TW	test water

4.3 Abbreviations

Al	aluminium
BOD_5	biochemical oxygen demand in 5 days
C	carbon
CH_4	methane
CO_2	carbon dioxide
COD	chemical oxygen demand
DS	dried solids
EPDM	ethylene-propylene-dien class M, a synthetic rubber material

Fe	iron
H_2S	hydrogen sulfide
MAP	magnesium ammonium phosphate (struvite)
ML	mixed liquor
MLSS	mixed liquor suspended solids
MLVSS	mixed liquor volatile suspended solids
N	nitrogen
NH_4	ammonium
NO_3	nitrate
N_2O	nitrous oxide (laughing gas)
orgN	organic nitrogen
O_2	oxygen
P	phosphorus
PE-HD	polyethylene with high density
PP	polypropylene
PT	total population
PVC	polyvinylchloride
RS	return sludge
SBR	sequencing batch reactor
TKN	total Kjeldahl nitrogen
TSS	total suspended solids
WWTP	wastewater treatment plant

5 Requirements

5.1 General

Biological reactors and final clarifiers are connected by return sludge recirculation lines and form a unit process: the activated sludge process. The performance of the process depends on biological and chemical reactions in the activated sludge tanks as well as separation of activated sludge in the final clarifiers. Activated sludge systems include structures, such as aeration basins and sedimentation tanks, and technical equipment, such as aeration systems and sludge scrapers.

Biological treatment and clarification (decanting) may be combined in a single sequencing batch reactor (SBR) with intermittent aeration and sedimentation.

The design shall take account of the requirements specified in EN 12255-1, EN 12255-10, EN 12255-11 and EN 12255-12.

Annexes A, B and C provide typical design values, typical wastewater characteristics and usual primary settling tanks' effectiveness.

5.2 Planning

5.2.1 Basic information

The design of an activated sludge system may be based on common values as provided in Annex B, in particular for plants serving up to 1 000 PT. For larger plants, the design should be based on the following information (ideally maximum or minimum 2 weeks average over 2 to 3 years):

1. Maximum and minimum wastewater temperature and temperature dependent requirements on the effluent quality;

2. Maximum, minimum hourly flow and yearly average wastewater inflow; and the maximum 2 h-inflow during dry weather conditions;

3. System loads, depending on primary treatment (where provided), including variations of COD (or BOD_5), TSS, P and TKN concentrations. The 85 %-quantiles should be provided for system design and the 50 %-quantiles (i.e. medians) or arithmetic averages should be provided for the calculation of operating costs and the design of sludge treatment facilities;

4. Where possible, the composition of the incoming COD shall be provided to the designer, separated into degradable dissolved COD, inert dissolved COD, degradable particulate COD, inert particulate COD and readily degradable COD; See Annex C for more information.

   NOTE With the standard methods, COD is analysed using dichromate as the oxidising agent. Chromium is a heavy metal. It would be more sustainable if dichromate could be replaced with a different oxidising agent.

5. A minimum of 40 samples should be analysed for all parameters. For plants serving less than 10 000 PT the number of samples may be less.

6. The consent standards concerning COD, N and P concentrations in the effluent.

Return loads from sludge treatment shall be taken into account, particularly ammonium return load. In some cases, it may be necessary to provide separate treatment of filtrate or centrifugate from sludge dewatering, e.g. using a de-ammonification process.

Load removal ratios during primary treatment shall be taken into account. It is recommended to investigate the removal ratios during dry weather conditions. Where this is not feasible, removal ratios as shown in Annex C may be used.

Biological treatment units should be protected from excessive hydraulic loads e.g. by the use of overflow devices and/or storm tanks to meet the required discharge consent. The frequency and volume of wastewater discharges should be limited (see EN 752).

If the waste water composition is unusual, it is recommended that a half-technical pilot test is performed for a minimum period of half a year (including the cold weather period) to investigate data for the system design. A design based on long-term testing can optimize the design and avoid safety factors necessarily included in a more general design.

Where the required sample analysis is not feasible, Annex A provides basic guidance information for system design.

The following factors shall be determined during planning of an activated sludge system:

- capacity and dimensions of the biological reactors;

- prevention of dead zones and of detrimental deposition in tanks/channels;

- establishment of multiple lines/units or other technical means to maintain the required final effluent quality while maintenance or repair work is carried out;
- aeration and/or mixing equipment in the biological reactors with sufficient capacity;
- surface area, volume and depth of final clarifiers;
- sludge removal system within clarifiers;
- sludge recirculation and surplus sludge wasting equipment;
- internal recirculation ratio and equipment;
- sufficient stabilization of the removed surplus sludge (where required);
- measurement and control systems;
- odour control;
- noise and vibration control;
- hydraulic head loss.

It may be necessary to add easily degradable organic carbon compounds (e.g. methanol) in order to achieve sufficient denitrification. Annex D provides information of such additives.

5.2.2 System selection

The configuration, number, shape and volume of reactors achieving the main biological reactions can vary considerably according to:

- plant size;
- the quality of treatment to be achieved, e.g. only BOD_5 (or carbon) removal, nitrification, denitrification and/or phosphorus removal;
- the requirement for simultaneous aerobic sludge stabilization (i.e. the required aerobic sludge age);
- selection of a single-stage or multi-stage system;
- where biological nitrogen removal is required: selection of the type of denitrification (e.g. pre-, cascade-, simultaneous, alternating, intermittent or post-denitrification);
- provision of anaerobic or aerobic selectors to mitigate sludge bulking;
- provision of anaerobic reactors to achieve enhanced biological phosphorus removal;
- provision of reactors which can use anoxic or aerobic treatment (depending on load and temperature);
- requirement for chemical phosphate removal by addition of metal salts (e.g. of ferric, ferrous or aluminium salts);
- minimum and maximum temperatures, and temperature dependent requirements (e.g. N-removal requirements).

Where biological nitrogen removal is required, nitrification and denitrification reactors shall be provided. Six systems can be distinguished (see Figures 2 to 7):

1. pre-denitrification in one or several anoxic reactors which are (usually) not aerated;
2. cascade denitrification with alternating anoxic and aerobic reactors whereby the inflow is fed to anoxic reactors;
3. simultaneous denitrification in a loop reactor (oxidation ditch) with alternating aerobic and anoxic zones;
4. alternating denitrification with parallel reactors that are sequentially aerated and non-aerated, whereby the inflow is always fed into the non-aerated reactor;
5. intermittent aeration providing for a sequence of aerobic and anoxic conditions within a reactor, e.g. in an SBR-reactor; intermittent aeration requires a substantially higher capacity of the aeration system;
6. post-denitrification with a carbon source fed into the anoxic reactor, followed by subsequent aeration (this system may be used where the C/N-ratio in the influent is so low that a carbon source shall be added anyway).

Pre-denitrification and cascade denitrification require recycling of nitrate containing wastewater from nitrification to denitrification reactors or zones. The internal recirculation ratio depends on the required denitrification ratio.

Enhanced biological phosphorus removal may be provided. It may offer the following advantages:

- saving of precipitants;
- reduced dry mass of surplus sludge;
- improved possibility of phosphorus recycling;
- lower reduction of the wastewater’s alkalinity depending on the precipitant used;
- lower concentration of anions (e.g. chloride) in the effluent.

Favourable conditions for enhanced biological P-reduction are:

- high ratio of readily degradable COD to the P-content in the influent;
- low oxygen and nitrate concentration in the flows entering the anaerobic reactor;
- if the flow pattern of the anaerobic reactor is close to a plug flow reactor or where it is a cascade reactor.

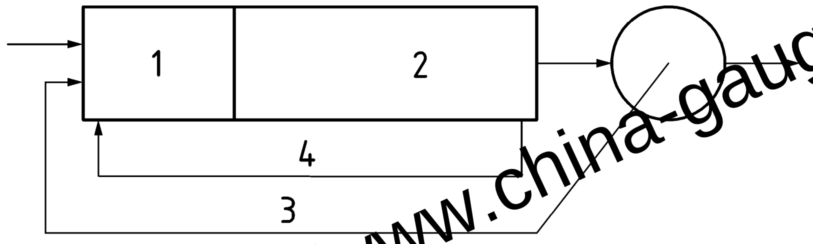
Disadvantages of enhanced Bio-P removal at plants with anaerobic sludge digestion are:

- Sometimes severe precipitation of struvite (MAP = magnesium-ammonium-phosphate) in anaerobic digesters and related equipment;
- Dissolved phosphate binds water, reduces the effectiveness of flocculants and impairs the dewatering results.

The addition of precipitants for P-removal is usually required. For this reason, the capability to add precipitant dosing facilities shall always be provided for even where they are not initially included.

Selection and design of the activated sludge system may be done with the help of dynamic modelling. This can be particularly helpful for the upgrading of existing systems.

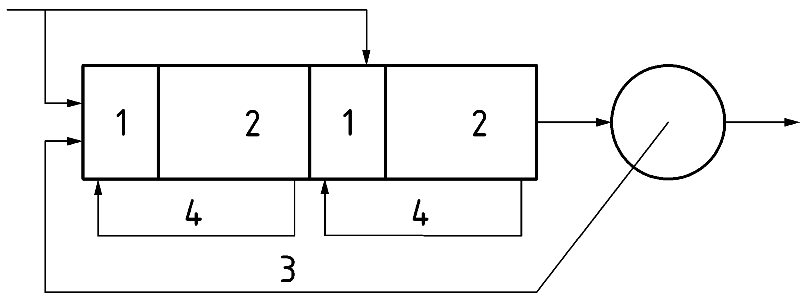
Figure 2 to Figure 7 show process options for nitrogen removal (nitrification plus denitrification) systems (Source: DWA-A 131).



Key

1 denitrification
2 nitrification
3 return sludge
4 internal recirculation (NO_3)

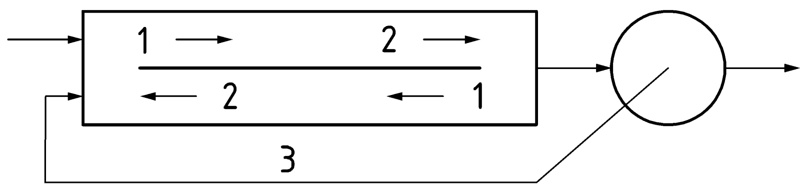
Figure 2 — Pre-denitrification



Key

1 denitrification
2 nitrification
3 return sludge
4 internal recirculation (NO_3)

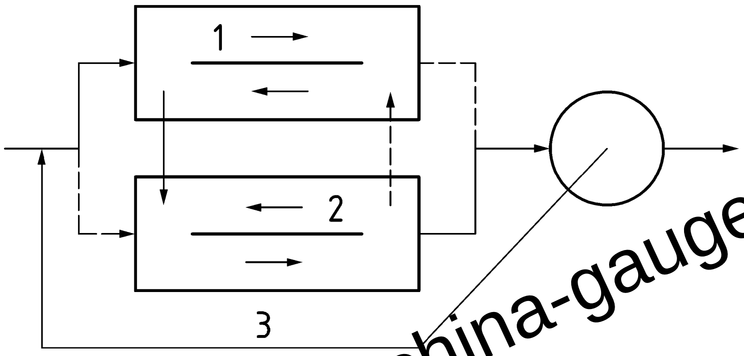
Figure 3 — Cascade-denitrification



Key

1 denitrification
2 nitrification
3 return sludge

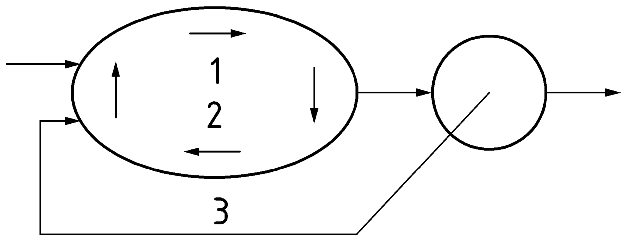
Figure 4 — Simultaneous denitrification



Key

1 denitrification
2 nitrification
3 return sludge

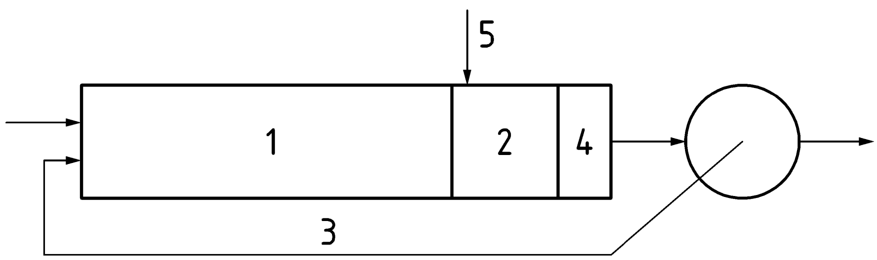
Figure 5 — Alternating denitrification



Key

1 / 2 denitrification or nitrification
3 return sludge

Figure 6 — Intermittent denitrification



Key

1 nitrification
2 denitrification
3 return sludge
4 post-aeration
5 organic carbon

Figure 7 — Post-Denitrification

Annex E provides information about the required sludge age depending on the requirements, Annex F about the surplus sludge production, Annex G about the required denitrification capacity, and Annex H about the oxygen consumption.

Annex I provides guidance how to determine the ratio of anoxic zones and Annex J about the reactor volumes.

Annex K shows how the internal recirculation ratio for N-removal is calculated.

Annex L shows how the required alkalinity is determined.

Annex M provides guidance on the design of aerobic selectors.

In Annex N the design with F/M-ratios is explained.

Annex O provides information on the sludge volume index.

Annex P explains how the concentration of the return sludge is calculated and Annex Q how the return sludge flow and its mixed liquor concentration are calculated.

5.2.3 Biological reactors

The process design shall be based on one of the following design parameters dependent on the required wastewater and sludge treatment quality:

- sludge age or
- sludge loading (F/M-ratio).

Both values depend on the concentration of the mixed liquor suspended solids (MLSS) or of the mixed liquor volatile suspended solids (MLVSS) which depend on the sludge volume index (SVI) and the performance of the final clarifiers.

The performances of biological reactors and final clarifiers are interdependent. For this reason, activated sludge systems shall be designed as a complete system.

Exemplary design information can be found in the informative Annex A and in the literature (see Bibliography).

It can be useful to provide an aerated or non-aerated selector in order to mitigate the development of sludge bulking (resulting from growth of filamentous bacteria, such as *microthrix* or *nocardia*).

Where intermittent pumping is provided, the influent and return sludge shall arrive at a selector at the same time.

Another or additional option to mitigate sludge bulking is the temporary use of aluminium salts instead of ferric or ferrous salts for phosphorus precipitation.

5.2.4 Clarifiers

Final clarifiers shall be provided with equipment for scum and foam removal.

Clarifiers shall provide for:

- separation of activated sludge solids from treated waste water by sedimentation;
- storing activated sludge to prevent it from overflowing during high hydraulic load;
- gravity thickening and removal of the activated sludge in order to recirculate it to the activated sludge reactor.

Each purpose requires a special zone in clarifiers. Information about the heights of the zones is provided in Annex S.

Sludge removal shall be slow at the bottom of the sludge thickening zone, preventing the generation of turbulence which could jeopardize sludge thickening.

Clarifiers can be upward flow (so called Dortmund tanks), horizontal flow or lamella separators (see EN 12255-4). Upward flow is limited to a surface area of about 100 m².

For general construction principles, the design of scrapers and their tracks, and for their design service life see EN 12255-1.

Sequencing batch reactor (SBR) systems do not need a subsequent clarifier and sludge return equipment because aeration, sedimentation and decanting occur intermittently within an SBR.

5.2.5 Environmental impact

Activated sludge systems can emit odour, noise and aerosols, particularly systems with surface aerators. Such impacts should be mitigated by structural means.

Even more problematic could be the emission of greenhouse gases. Activated sludge systems require much energy (50 % to 80 % of the entire WWTP). They oxidize carbon matter to carbon dioxide which is released into the atmosphere. They also emit small quantities of the very strong greenhouse gases nitrous oxide (N_2O) and methane (CH_4).

Environmental effects should be considered before selecting a system.

5.3 Detailed Design

5.3.1 Flow-splitting

When the process involves multiple lines or parallel units, the incoming flow shall be distributed by adjustable distribution devices (e.g. weirs, gates, or valves) that can also be used to isolate each treatment unit.

Accumulation and removal of floating matter shall be considered during planning of flow-splitting devices.

5.3.2 Biological reactors

Biological reactors can be completely mixed. Continuous flow reactors can also be designed to achieve sequential reaction characteristics (e.g. close to plug flow). This can either be achieved by a series of several mixed reactors (cascades) or by providing long reactors with a length to width ratio of about 15 : 1.

Reactors provided with fine bubble diffusion should have a minimum depth of 4 m. If such reactors are more than 6 m deep, means for removing gas from mixed liquor should be provided between the biological reactors and final clarifiers.

A minimum of three reactor zones shall be provided for systems with pre-denitrification, whereof the second can be operated anoxic or aerobic, depending on temperature and load. This requires installation of both aeration and mixing equipment.

At least two parallel reactors or subsequent cascades should be provided for all WWTPs serving more than 10 000 PT. Each reactor shall be provided with isolation means. A bypass shall be provided for smaller plants.

Where SBR reactors are used, at least two SBR reactors should be provided for plants serving more than 1 000 PT to equalize flow and aeration if they are not continuously fed.

If the plant is designed for one or more reactors to be taken out of service for routine maintenance, the reactors remaining in operation and their associated pipework, channels, etc., shall have the capacity to accommodate the design wastewater flow and ensure the required effluent quality.

NOTE Local or national regulations can permit this to be temporarily less stringent than in normal operation in certain conditions.

Tanks shall be designed to allow emptying either by gravity flow or by pumping. Structures shall be designed such that emptying will not affect their stability, irrespective of the groundwater level. All necessary measures shall be taken, such as ballast concrete or facilitating lowering of the groundwater level, in order to prevent flotation.

The floors of tanks should slope towards a low point to facilitate emptying. When a pump is used for emptying, a drain pit should be provided at the low point.

The hydraulic design shall minimize short-circuiting. Completely mixed reactors are also possible. It is desirable for the flow through reactors to be close to a plug flow. This can be achieved in rectangular tanks with a meandering flow pattern or a partial blocking of the longitudinal flow.

In the case of a multipoint feed system (e.g. step-aeration), appropriate devices (e.g. weirs, gates or valves) shall be provided to allow modification of the original flow-splitting arrangement. The same applies to systems where the return sludge is fed at various points.

The water level in biological reactors can be controlled by fixed or adjustable overflow weirs.

The freeboard of aeration tanks shall be sufficient to prevent overflowing of mixed liquor or scum (or foam) under normal operational conditions. A freeboard of 0,5 m should be provided.

Foam of varying stability and viscosity can develop, particularly where filamentous bacteria are prevalent. The number of possible points of accumulation shall be minimized. In addition, bottom openings shall be provided in walls separating compartments in reactors to prevent high water pressure on the separating walls when the reactor is emptied.

All emissions from reactors shall comply with national requirements.

Where reactors are covered (e.g. for environmental reasons), the materials used shall be capable of withstanding the aggressiveness of the atmosphere which shall be especially taken into account where septic wastewater (H_2S-corrosion) or aggressive industrial effluents can arrive. In such cases, the walls above the water level shall also be protected down to 0,3 m below the lowest operating water level. It shall also be considered that ammonia can be corrosive to stainless steel. Means for explosion prevention should be considered.

In such cases, forced ventilation can be used to limit the aggressiveness of the atmosphere and increase the service life of structures and equipment. Forced ventilation shall be installed, if staff need to enter the enclosed space.

5.3.3 Mixing

The mixed liquor in all reactors shall be agitated to prevent activated sludge from settling or forming detrimental deposits. The design of tanks, piping and mixers should avoid short-circuiting of flows that need to be mixed throughout the tank volume, e.g. incoming wastewater, returned activated sludge and mixed liquor recirculation. Mixing can be performed with:

- aeration systems or equipment (e.g. fine-bubble diffuser systems or surface aerators);
- coarse bubble diffusion for the generation of a rotational flow patterns (with little oxygen transfer);
- mechanical mixing (e.g. with propeller or jet mixers).

A combination of several systems is possible.

Intermittent or variable speed operation of mixers can save energy.

Mechanical mixers shall be designed to minimize debris accumulation and cording by fibrous materials. Mechanical mixers shall be removable without emptying the tank.

All mixing systems shall be designed such that they are capable of preventing the generation of solid deposits and that they are capable to re-suspend settled solids. Where no primary clarifiers are provided, the design of mixing systems depends on the effectiveness of preceding grit chambers (see EN 12255-3).

The choice of mixing system depends on the characteristics of the wastewater to be treated, the geometric reactor configuration and potential short-circuiting. The electrical power consumption of mechanical mixers is typically between 1 W/m³ and 5 W/m³. A power consumption as low as 0,3 W/m³ may be sufficient if the wastewater pre-treatment, geometry of the tank and the design of the mixer are optimized.

The power consumption of mechanical mixers is a poor indicator of mixing performance. The momentum generated by the mixer, in combination with the installation geometry, is a better criterion for the mixing result. For axial flow submersible mixers, momentum generation is given by the mixer thrust according to ISO 21630:2007. For long shafted axial mixers the same principle can be applied.

The momentum required to drive the mixing flow can be generated by large and slow propellers more energy efficiently than by small and fast propellers. Propeller design and motor efficiency at the operating points have an additional impact on energy efficiency. Large propeller mixers usually require strong fixation because they produce strong thrust and torque. Large propellers can be more sensitive to upstream turbulence.

Propeller mixers subject to fluctuating thrust or torque could lose their intended performance and suffer fatigue. Such fluctuations occur if air bubbles or skewed water currents enter the propeller suction zone. To avoid this, sufficient clearance between propeller and aeration equipment in all directions shall be ensured by the designer or by the system supplier (if the latter supplies the aeration equipment and the mixers). The designer shall avoid placing the propeller in strong or skewed flow, e.g. caused by pipe or weir inlets, baffles or guide vanes, or other mixers.

Slow running propellers should be preferred to prevent destruction of flocs.

The power input of fine or coarse bubble diffusion systems, calculated as isothermal decompression power shall be minimum 2 W/m³:

$$P / V = Q_1 \cdot p_1 \cdot \ln\left(p_1 / p_2\right) \geq 2 \text{ W} / \text{m}^3 \quad [\text{W/m}^3] \qquad (1)$$

where

- P is power released during decompression in W;
- V is tank volume in m³;
- Q_1 is air flow at p_1 in m³/s;
- p_1 is pressure at the immersion depth in Pa;
- p_2 is atmospheric air pressure in Pa.

This requirement is usually met when the air flow of fine or coarse bubble diffuser systems is minimum 1 Nm³/h per m² surface area. The power consumption of the blower is far higher than the decompression power P.

Installation of coarse bubble diffuser pipes is a simple method for mixing denitrification reactors. Intermittent mixing can save energy.

More powerful mixing with about double the energy input, is required where the activated sludge process is not preceded by primary treatment or by excellent preliminary treatment (particularly effective grit chambers).

The designer shall verify that the power input of fine bubble diffuser systems at their minimum air flow, is not lower than the power required to ensure adequate mixing. However, the air flow could be intermittently increased for sufficient mixing.

5.3.4 Aeration

5.3.4.1 General

Designers shall consider the variation in oxygen consumption that will arise between and within reactors.

The designer shall calculate and specify the oxygen transfer rate in clean water for the following load conditions for each reactor:

- Average present load;
- Maximum present load;
- Minimum present load.
- Maximum future load.

The designer shall specify the α-factor of the mixed liquor and of the specified aeration system (e.g. fine bubble diffusers). This may require prior testing. The α-factor also depends on the type of aeration system.

The designer shall also specify the following on-site characteristics for which the system is to be designed:

- atmospheric pressure;
- air temperature;
- air humidity.

Aeration systems or equipment shall be sufficient for the required maximum oxygen transfer per hour OC_h (in kg O_2/h) to each reactor. They shall also be capable to provide the minimum oxygen transfer rate, whereby intermittent aeration may be used.

Above 1000 PT aeration systems should have redundancy, either built-in or kept in store.

Aeration systems shall be designed to operate under the most severe on-site conditions (e.g. extreme temperatures and humidity).

Unless otherwise agreed, the design service life (see EN 12255-1) of the equipment for aeration shall be:

- Class 5: for gears and bearings of surface aerators;
- Class 3: for all electrical motors;
- Class 4: for mixers and blowers.

Further information on the design of fine bubble diffusor systems is provided in the informative Annex W.

Part 15 of this standard series provides information on the testing of the capacity and efficiency of aeration systems with clean water.

There are other methods for testing the system compliance, e.g. off-gas analysis during operation or a long-term balancing of the in- and outflows of COD and Nitrogen. Such testing methods can only certify, but not disapprove, the efficiency of aeration systems. In the case that an aerations system does not reach the desired performance, a clean water test shall be performed. The reason for this limitation is that the supplier of an aeration system cannot be made responsible for the alpha-factor of the mixed liquor.

The dissolved oxygen concentration in aerated reactors or aerated zones should be maintained between 1,0 mg/l and 2,0 mg/l, and in nitrifying reactors at about 2 mg/l.

5.3.4.2 Fine bubble diffuser systems

Fine-bubble diffuser systems have become dominant in Europe. Coarse bubble diffuser systems are not energy efficient and are thus seldom installed in Europe.

Fine bubble diffuser systems have an α-value between 0,35 and 0,9. The α-value is especially low where the MLSS concentration or the F/M-ratio is high.

Table 1 may be used as a guideline for the selection of α-values for fine bubble diffuser systems.

Table 1 — Typical α-values depending on process type and load [source: DWA-M229–1]

Type of process	α at minimum load	α at average load	α at maximum load
C removal only	0,6	0,5	0,35
N removal (nitrification + denitrification)	0,85	0,75	0,6
SBR systems with N removal	0,8	0,65	0,5
Simultaneous aerobic stabilization	0,9	0,8	0,7

Because the range from the minimum to the maximum oxygen transfer rate is often wide (e.g. 1 : 8), and because the air flow through fine bubble diffusers is typically limited to a range of 1 : 6, aeration systems may be operated intermittently which requires membrane diffusers that can be shut off without the risk of clogging.

Fine bubble diffuser systems should generate air bubbles with a diameter of about 2 mm. Though finer bubbles have a larger surface per volume and a lower rising velocity, they have a stable boundary layer of water attached requiring diffusion through it. Coalescence of fine bubbles to larger bubbles cannot be prevented.

Fine bubble diffuser systems should be spread above the tank's bottom to prevent generation of fast vertical water flow above clustered diffusers, reducing the retention time of the bubbles. Coverage of the bubble release area is typically 10 % to 30 % of the bottom area. It is higher in high-loaded reactors or where an especially high energy efficiency is required, but may be lower in systems for simultaneous aerobic sludge stabilization. Such systems are often designed as loop reactors whereby aerated sections are followed by non-aerated sections.

Suppliers of diffuser systems shall provide their proposals for their system's layout and guarantee values for the Standard Oxygen Transfer Rate (SOTR) and Standard Oxygen Transfer efficiency (SOTE) for maximum, average and minimum loads as required (see EN 12255-15).

Small clusters of diffusers should be avoided in loop reactors. They generate not only a strong upward velocity above the diffuser area, but also form a break to the circular flow. The standard testing method for the Oxygen Transfer Rate (SOTR) and Standard Oxygen Transfer efficiency (SOTE) might not be applicable because the reactors are not sufficiently mixed. EN 12255-15 includes as an informative annex a test method for loop reactors with inadequate mixing.

The lower the air flow per diffuser, the higher is the diffuser density, and he deeper the reactors, the better is the standard oxygen transfer efficiency in clean water (SOTE, see EN 12255-15).

Air diffusers shall be installed with a vertical tolerance of maximum ± 6 mm to ensure even air distribution. Their level can be adjusted while the reactor is filled with clean water to the level of the diffusers.

Membrane diffusers are frequently used. In comparison to ceramic diffusers they have the advantage that they can be shut off without the danger of clogging. Membranes made of EPDM are most common. Silicon or polyurethane membranes are also available for special applications.

The number of diffuser grids, defined as having one supply pipe and a shut-off valve, per reactor shall be specified.

Diffuser systems shall have equipment for the removal of condensed water. This is usually a small pipe with a manually operated ball valve from the lowest point of the diffuser grid to about 20 cm above the water level. Alternatively, a continuous condensate removal system may be used.

Fine-bubble diffuser systems should permit dosage of formic acid into the air flow. The formic acid serves for removing precipitations, e.g. calcium carbonate, from the slots of the membranes and thus can reduce the diffusers' head loss.

Blowers, supplying compressed air to the diffuser system, shall be designed such that they can provide the minimum and maximum air flow. Variable frequency drives are commonly used today. The curve of blower efficiency over its air flow rate depends on the type of blower selected (e.g. roots, screw or turbo blowers). It is recommended the blowers are sized such that their average yearly power consumption is minimized. The maximum required air supply shall be available with any one blower out of service.

Blowers shall be provided with adequate noise and vibration protection, dependent on local requirements.

Each blower shall be provided with an isolation valve on its pressure pipe.

The air entering the blowers shall be filtered to remove dust and oil because such matter could cause blockage of the diffusers.

The compressed air may be warm or hot, depending on the pressure increase by the blowers. However, the air pipelines should not be thermally insulated because the air flow decreases as the air is cooled and because air which is too hot could damage membrane diffusers. Protection against touching the pipelines may be required for safety reasons.

Where the immersion depth of the diffusers in all reactors is equal, there should be a common manifold for all blowers. This provides maximum flexibility of blower operation and reduces redundancy requirements.

Air supply pipelines should be made of an appropriate grade of stainless steel because they require little maintenance and have a long life. Pipelines above water should be stainless steel CrNi18-9, e.g. 1.4307, AISI 304. Where the chloride concentration is high, pipelines below water might need to be stainless steel CrNiMo 17-12-2 e.g. 1.4404, AISI 316L. Because stainless steel pipelines have a small wall thickness of 2 mm to 3 mm, there is a danger of resonance vibrations causing noise. For this reason and to prevent high pressure losses, the air velocity in supply pipelines shall not exceed 15 m/s, but the velocity in short drop pipes to diffuser grids may exceed 15 m/s. The velocity in distribution pipes shall be limited to 8 m/s to guarantee even flow distribution.

PE-HD or PP pipelines may also be used; their pressure rating should be 0,6 MPa.

The pressure loss in the air pipelines should not exceed 2 to 3 kPa at maximum flow. An additional 2 to 3 kPa, depending on the diffuser type, is usually needed to overcome the head loss of the diffusers at maximum flow. An additional 2 kPa may be required due to gradual diffuser aging or fouling.

The sum of the head losses (typically 7 kPa) and the water pressure at the immersion depth at maximum water level determines the minimum pressure increase by the blowers.

An example for the design of a fine bubble diffuser system is provided in Annex W.

A certain distance from diffusers to mechanical mixers is required. A very strong water current could damage diffusers. However, it is not possible to provide general guidelines. The diffuser supplier shall give guidance of the maximum water velocity the diffuser installation can withstand. The designer shall ensure that diffusers are not subject to higher water velocities generated by mechanical mixers.

5.3.4.3 Surface aerators (vertical shaft and brush aerators)

Surface aerators generate aerosols and splashing noise. Adequate protection is required, depending on the environment. Although vertical shaft and brush aerators have an α-factor of 0,8 to 1,1, their energy efficiency is lower than that of fine bubble diffuser systems, but they are less expensive to install and easier to maintain except:

- in freezing climates;
- where wind brings sand or other particles;
- where hot climates degrade their lubrication.

Vertical shaft surface aerators are usually installed in square tanks with a depth of up to 4,5 m. The length of the sides of the tank should be at least 4 times the depth. Their oxygen transfer rate and power consumption are controlled through variation of the water level and speed. Such aerators can also be installed at the far ends of a loop reactor (at the ends of the dividing wall) where they also generate a circulating flow through the loop reactor.

Where vertical shaft surface aerators are used, consideration shall be given to the prevention of cavitation eroding the bottom surface of the tanks. Protecting steel plates are usually provided.

Vertical shaft surface aerators have usually a diameter between 1,5 m and 4 m. Their circumferential velocity is usually 4 m/s to 6 m/s. To mitigate excessive generation of aerosols, they shall be provided with a circumferential skirt.

Brush aerators with horizontal shafts are designed for both aeration and circulation in loop reactors. The distance between subsequent brush aerators should be minimum 20 m. The distance of a brush aerator from the end of a straight channel should be the channel width or 4,5 m, whichever is greater.

The depth of loop reactors with vertical shaft and brush aerators is limited to about 2,5 m to provide sufficient bottom flow velocity. If they are provided with brush aerators and suitable guide baffles, they may be up to 3,5 m deep. However, they may be deeper if additional propellers are provided to generate a sufficient horizontal flow velocity.

Simultaneous denitrification can be achieved in loop reactors with brush aerators because the lower zones have a low oxygen concentration. However, it is difficult to control the process.

Brush aerators usually have a diameter of 1 m and a length of up to 9 m. Their immersion depth is maximum 0,3 m. Their circumferential velocity is usually between 3 m/s and 4 m/s. The shaft is driven by a motor via an elastic torsion coupling.

In order to mitigate noise and aerosol emission, brush aerators shall be covered. Air enters and leaves through gaps between the cover and the water surface. A guide plate shall be installed on the downstream side of brush aerators, guiding surface water to a lower depth. The guide plate can increase the efficiency of brush aerators by up to 20 %.

The utilization factor (see EN 12255-1) for the layout of gears and bearings of vertical shaft and brush aerators shall be $f_U = 2$. Rotor blades and main shafts shall be designed to the fatigue strength at nominal load. The maximum deflection of shafts on horizontal aerators caused by load and weight shall be less than 1/1 000 of the shaft’s length.

5.3.4.4 Injectors or ejectors

Injectors draw air through a bore in their shaft driving a propeller. They are useful for small applications due to their low weight and flexibility. They can also be used for sludge aeration and mixing. They have a rather low energy efficiency.

Ejectors draw air into a pumped flow. The flow is typically generated by a submerged pump. They have a venturi type nozzle which draws atmospheric air through a pipe and mixes the air with the liquid. The air flow can be boosted with a blower. Air-liquid mixture is ejected into the liquid, providing oxygen transfer and mixing. Submersible ejectors are suitable for quick installation without tank drainage.

5.3.5 Secondary clarifiers

At least two parallel clarifiers should be provided for WWTPs serving a total population of more than 20 000. The sum of the capacities of the clarifiers shall be at least 100 %. However, each clarifier shall be hydraulically able to handle a higher flow to accommodate the possibility that one clarifier may be out of service. Each clarifier shall be provided with isolation means so that it can be taken out of service. A bypass shall be provided for plants with a single clarifier.

Degassing structures may be used upstream of clarifiers to improve their performance by removing gas bubbles from the mixed liquor, especially where aeration tanks are deep. Such structures, where provided, shall be installed between the last biological reactor and the clarifier. They are also an appropriate location to remove floating matter.

In addition to the type of process planned and the required efficiency of separation, the sizing parameters also depend on the type of clarifier and notably the minimum settling rate. This rate depends on the specific hydraulic characteristics of both upward flow and horizontal flow clarifiers and whether or not the selected process is equipped with lamella modules.

The design shall be based on both:

- the surface flow rate; and
- the surface loading rate.

In addition, the clarifiers' surface and depth depend on the following:

- characteristics of the sludge, expressed as sludge volume index (SVI) or stirred sludge volume index (SSVI);
- concentration of the incoming mixed liquor (MLSS);
- the type and shape of the clarifier.

NOTE Every g of suspended solids in the final effluent has a COD of about 0,8 g to 1,4 g.

Three zones can be distinguished in a clarifier (see Annex S):

- clear water zone;
- transition zone;
- gravity thickening and sludge removal zone.

The inflow zone shall ensure:

- dissipation of inflow energy;
- even distribution;

- degassing;
- flocculation of the biomass.

The sludge storage zone is designed to contain a sufficient sludge inventory, which is especially important to cope with a surge inflow requiring an increased sludge return flow. This is especially important where combined sewer systems provide high flow and load variations requiring a proportional increase of the return sludge flow.

The sludge thickening zone shall be designed to provide for return sludge with high solids concentration.

Small and deep clarifiers (Dortmund tanks), with a diameter not exceeding 15 m, do not require a scraper. Their bottom slope shall be not less than 60° (for pyramidal shaped sludge hoppers this slope applies to the edge between straight surfaces).

Circular clarifiers shall have a maximum diameter of 50 m. Where blade scrapers are used a bottom slope of minimum 7° is required. Large diameter clarifiers can be subject to disturbance by strong wind.

For larger clarifiers with a flat or slightly inclined floor, a sludge removal device is necessary, such as one of the following types:

- rotating scraper blades moving the settled sludge towards a hopper at the clarifier’s centre (circular or rectangular clarifiers);
- scraper bars, connected to chains, moving the sludge to a sludge hopper at the inflow end (only rectangular clarifiers);
- suction devices fixed to travelling bridges evacuating the sludge from the bottom of the tank (circular or rectangular clarifiers).

Overflow can occur over weirs with a maximum flow rate not exceeding 20 m^3/h per m weir length, or 10 m^3/h per m for overflow channels with a weir on each side. Overflow weirs shall have a serrated shape to distribute the flow. Scum baffles are usually provided to prevent scum overflow.

The outer rim of circular clarifiers is typically provided with an overflow weir. One-sided or two-sided overflow weirs may be provided. The length of the end wall of rectangular clarifiers is usually too short for this. For this reason, overflow weirs may also be provided along some distance of the side walls close to the end wall. Alternatives to overflow weirs include submerged pipes with perforations through which the effluent flows or submerged channels. The pipes end in chambers with an overflow weir, defining the water level. Scum cannot enter such submersed pipes. Another advantage is that they can be arranged to remove effluent from a large area, though they shall not impede a scraper’s operation.

The sludge removal equipment shall be designed to ensure sufficient sludge recovery, to maintain the required MLSS and to avoid anoxic conditions. The velocity of scrapers shall be low enough to minimize turbulence.

Means for the collection and removal of floating sludge and scum shall also be provided. The scum should flow into a surplus sludge tank provided with a mixer.

In circular clarifiers a circular inlet structure shall be provided to reduce the flow velocity. The inflow velocity into clarifiers shall not exceed 7 cm/s.

Variable level inflow equipment permits adjustment to the sludge level.

The slope of sludge hoppers shall be minimum 60°.

Automated cleaning of the overflow channels and baffles may be specified, e.g. with brushes or spray water.

Further information on the design of clarifiers is provided in Annex R, Annex S, Annex T, Annex U and Annex V. Construction principles can be found in EN 12255-1

5.3.6 Return and surplus sludge systems

The return sludge system returns sludge from clarifiers back to reactors in order to maintain the mixed liquor concentration required for the biological process.

The system shall be designed to allow variation of the return sludge flow. The range of the return sludge flow shall be between 50 % and 100 % of the maximum wastewater inflow during wet weather. A minimum of two variable speed pumps shall be provided, one of these being a standby pump. Another option is a pair of Archimedian screw pumps which do not require variable speed drives.

The system should be designed to minimize aeration of the return sludge where it is returned to an anaerobic or an anoxic zone.

The system shall be provided with standby pumps, which may be a portable pump in the case of small treatment plants.

Surplus sludge generated during the biological process shall be removed in order to maintain the MLSS concentration in the biological reactors at a level preventing overflow of biomass from the final clarifiers.

The mass and volume of sludge to be removed from the system depends primarily on the wastewater composition, the type of process, the sludge age and the sludge volume index (SVI) or stirred sludge volume index (SSVI). In addition, it depends on the design of the final clarifier.

At least two surplus sludge pumps shall be provided (where surplus sludge pumps are required), of which one is for standby. The capacity of each shall be minimum 120 % of the calculated surplus sludge production. Variable speed pumps shall be provided where the surplus sludge is pumped to mechanical sludge thickening.

Surplus sludge is usually removed from the return sludge.

Surplus sludge may be returned to primary clarifiers for co-thickening with primary sludge; however, separate (mechanical) thickening of surplus sludge should be preferred.

Surplus sludge storage shall be kept to a minimum where enhanced biological P-removal is employed because phosphate is released rapidly if the sludge becomes anaerobic. In this case gravity thickening of surplus sludge cannot be used, mechanical thickening is required.

Scum shall not be returned to aeration tanks, but shall be blended with surplus sludge. A scum breaker may be required in the surplus sludge storage tank or thickener.

5.3.7 Internal recirculation

Pumps for internal recirculation only need to generate low head. Propeller pumps are often used. The flow should be regulated by variable speed drives or throttling devices, controlled by nitrate measurement in the effluent.

5.3.8 Control and automation

National or local regulations or the relevant authority can set requirements for monitoring and control.

For WWTPs serving a total population equivalent exceeding 1 000 PT, the oxygen concentration in each reactor shall be measured and used for controlling the oxygen supply by the aeration system or equipment.

Loop reactors shall be provided with several oxygen concentration probes.

Systems for simultaneous aerobic sludge stabilization shall be designed for an aerobic sludge age of minimum 20 days at design temperature (usually between 10 °C and 12 °C). The control system shall maintain an oxygen concentration in the aerobic reactors or zones of about 1,5 mg/l.

Operators should not try to reduce power consumption of systems for simultaneous aerobic stabilization below the above limit. They may still achieve the effluent requirements, but their surplus sludge will not be stabilized, is odorous and has poor dewaterability. Such operators usually pay more for sludge disposal than they save in power consumption.

Blowers of fine bubble diffuser systems serving several reactors require flow control in the pipelines to each reactor. The control system shall be designed such that at least one flow regulator is fully open.

Auto-rotation of the blowers shall be provided.

It is usually not necessary to measure air flow rates. The power consumption of each blower shall be transmitted to the central control station.

The control system should include provision for automatic air flushing of diffusers, i.e. increasing the air flow to about 125 % of the design flow for a few minutes per day. This may require the possibility to shut off the air flow to one reactor in order to provide sufficient air flow for flushing another reactor.

Systems for nitrogen removal should have online-measurements of the ammonium and nitrate concentrations in their effluent, controlling the aeration system and the internal recirculation flow.

Sensors for the mixed liquor concentration in reactors and sludge blanket control in final clarifiers may be required. They are used to control the sludge return and surplus sludge removal pumps.

A sensor detecting the rotation speed of a not driven wheel of a scraper may be requested to monitor the scraper’s function.

Systems shall be automated such that they operate reliably and efficiently without operator attendance.

Automatic control systems should be designed in such a way that the processes are coordinated with one another as comprehensively as possible.

The automatic control system should be implemented in such a way that changes in the process management can be incorporated flexibly in the control system.

Overflow weirs shall be readily adjustable. Overflow weirs of reactors with surface aerators at constant level shall be adjusted automatically, controlled by the oxygen concentration in the reactor.

The system shall be designed to change into a fail-safe mode in case of a failure of the automated control system.

A surge flow can be amplified by hydraulic and load feedback in systems with surface aerators at a constant level. The water level and the oxygen demand typically rise simultaneously. The latter leads to an additional raising of the overflow weir. The combination of these effects can jeopardize the function of surface aerators at a constant level.

The following information shall be recorded at WWTPs serving a total population of minimum 1 000:

- wastewater flow rate;
- temperature of the wastewater;
- oxygen concentration in all reactors;
- pH of the influent and effluent;
- speed and power consumption of each blower.

In addition, it is useful to monitor or record the following information at WWTPs serving a total population of minimum 1 000:

- MLSS concentration;

- ammonium and nitrate concentrations of the effluent;
- position of automatic valves;
- pressure in the blower manifold;
- turbidity of the effluent;
- position of automatic control valves;
- power consumption of each mixer;
- return sludge flow;
- scraper movement.

A first-priority alarm shall be raised, if:

- sludge return fails;
- aeration or mixing fails.

A second-priority alarm shall be raised if:

- surplus sludge removal fails;
- the oxygen concentration in a reactor breaches a minimum or maximum setpoint;
- the ammonium or nitrate concentration, or the turbidity in the effluent exceeds a maximum setpoint.

A first-priority alarm demands immediate action, a second-priority alarm requires routine action.

6 Test methods

See EN 12255-15 for testing of the Standard Oxygen Transfer Rate (SOTR) and of the Standard Oxygen Transfer Efficiency (SOTE) in clean water.

Annex A
(informative)

Design of biological reactors

Table A.1 shows typical design values. However, it should be noted that such values are temperature dependent [20].

Table A.1 — Typical process design parameters

Required treatment	**Sludge age (MSRT)**[a] d	**F/M-ratio (BOD_5)**[a] kg/(kg·d)
Partial treatment	≤ 2	≥ 0,5
COD-removal	2 to 4	0,25 to 0,5
Nitrification[b]	7 to 10	0,1 to 0,15
N-removal[b, c] (nitrification + denitrification)	10 to 15	0,07 to 0,1
Simultaneous aerobic sludge stabilization[b, c, d]	25 to 30	0,03 to 0,05

a The ranges apply for 10 °C or 12 °C design wastewater temperatures.

b Where enhanced biological phosphorus removal is desired, a preceding anaerobic contact period of 0,5 h to 2 h is required.

c For nitrogen removal 20 % to 60 % of the reactor volume may be anoxic, depending on the load and temperature.

d For sufficient aerobic stabilization the aerobic sludge age shall be minimum 20 d at design temperature.

Annex B
(informative)

Raw wastewater characteristics

It is recommended that the characteristics of the raw wastewater arriving are analysed. The following inflow characteristics may be used for municipal wastewater where no measured data are available. The following values are 85-percentiles of the incoming loads and not median or average loads which should be smaller. It is assumed that the loss on ignition, i.e. the ratio of volatile solids to total solids in the inflow is 80 %.

- $l_{COD,in}$: 120 g/(P·d)
- $l_{COD,dis,inert,in}$: 0,05 · 120 g/(P·d) = 6 g/(P·d)
- $l_{COD,part,inert,in}$: 0,3 · 120 g/(P·d) = 36 g/(P·d)
- $l_{COD,inert,in}$: (6 + 36) g/(P·d) = 42 g/(P·d)
- $l_{COD,deg,in}$: (120 – 42) g/(P·d) = 78 g/(P·d)
- $l_{COD,redeg,in}$: 0,2 · 78 g/(P·d) = 16 g/(P·d)
- $l_{TSS,in}$: 70 g/(P·d)
- $l_{TSS,inorg,in}$: 0,2 · 70 g/(P·d) = 14 g/(P·d)
- $l_{TSS,org,in}$: (70 – 14) g/(P·d) = 0,8 · 70 g/(P·d) = 56 g/(P·d)
- $l_{COD,part,in}$: 1,6 kg COD / kg TSS · 0,8 · 70 g/(P·d) = 90 g/(P·d)
- $l_{COD,dis,in}$: (120 – 90) g/(P·d) = 30 g/(P·d)
- $l_{COD,dis,deg,in}$: (30 – 6) g/(P·d) = 24 g/(P·d)
- $l_{TKN,in}$: 11 g/(P·d)
- $l_{P,in}$: 1,8 g/(P·d)

NOTE Local conditions can provide different sets of inflow data. Wastewater with a substantial portion of commercial or industrial influent might have different characteristics.

The indices stand for:

- COD: chemical oxygen demand;
- TSS: total suspended solids;
- TKN: total Kjeldahl nitrogen;
- P: phosphorus;
- in: inflow;
- inert: inert, not degradable;
- deg: degradable;
- redeg: readily degradable;
- dis: dissolved;
- org: organic;
- inorg: inorganic;
- part: particulate.

Annex C
(informative)

Removal efficiency of primary clarifiers

The removal efficiencies of primary clarifiers are shown in Table C.1. See EN 12255-4 repeated here for convenience.

Table C.1 — Removal efficiencies of primary clarifiers (DWA-A 131 modified)

	Removal effectivity η in %				
Retention time in primary clarifier calculated with the average dry weather flow	**0,75 h to 1 h**	**1 h to 1,5 h**	**1,5 h to 2 h**	**2 h to 2,5 h**	**> 2,5 h**
COD	30 %	32,5 %	35 %	37,5 %	40 %
Particulate COD	45 %	50 %	55 %	57,5 %	60 %
TSS	50 %	55 %	60 %	62,5 %	65 %
TKN	10 %				
P_{tot}	10 %				

Annex D
(informative)

External carbon sources

Where the COD/TKN-ratio is insufficient for reliable denitrification (where required), usually if it is below 6 : 1, and more specific, where the $l_{COD,redeg,in}/l_{TKN,in}$-ratio is below 1 to 1,2, it may be necessary to add readily degradable COD as a carbon source (e.g. methanol). Table D.1 shows characteristics of some carbon sources.

Table D.1 — Characteristics of carbon sources for denitrification (DWA-A 131)

Parameter	Unit	Methanol	Ethanol	Acetate
Density	kg/m^3	790	780	1 060
m_{COD}/m_{Dos}	kg/kg	1,50	2,09	1,07
m_{COD}/V_{Dos}	kg/m^3	1 185	1 630	1 135
$Y_{COD,Dos}$	kg/kg	0,45	0,42	0,42

m_{Dos} is the mass in kg and V_{Dos} is the volume in m^3 of the carbon sources. $Y_{COD,Dos}$ is the yield of additional kg COD in the biomass per kg COD of the dosed carbon source.

Annex E
(informative)

Sludge age (MSRT) and aerobic sludge age (MASRT)

E.1 System for Carbon Removal only

The aerobic sludge age *MASRT* of plants without nitrification should be 4 to 6 days dependent on the plant's size and the design temperature T_{des} which is typically 10 °C or 12 °C.

E.2 Systems for nitrification at minimum 12 °C

The aerobic sludge age *MASRT* of plants with nitrification, but without denitrification, shall be

$$MASRT \geq f_{Proc} \cdot 3,4 \cdot 1,103^{\left(15-T_{des}\right)} \quad [\mathrm{d}] \tag{E.1}$$

where

f_{Proc} is a dimensionless process factor between 1,2 and 2,4 depending on the plant's size, variations of flow, load, temperature, requirement for nitrification and pH value;

T_{des} is the design temperature in °C depending on ammonium reduction requirements. Where the wastewater temperature during the cold season may drop below T_{des}, a by 2 °C to 4 °C lower T_{des} shall be used in Formula (E.1) in order to guarantee sufficient nitrification capacity when the wastewater temperature has risen to T_{des}. The temperature difference is proportional to the velocity of the temperature increase at the end of the cold season;

E.3 Systems for nitrification and denitrification at minimum 12 °C

Where nitrification and denitrification are required, the sludge age *MSRT* should be

$$MSRT \geq f_{Proc} \cdot 3,4 \cdot 1,103^{\left(15-T_{des}\right)} \; / \left[1-\left(V_{Den} / V_R\right)\right] \quad [\mathrm{d}] \tag{E.2}$$

where

V_{Den}/V_R is the ratio of the anoxic denitrification volume V_{Den} to the total reactor volume V_R. This ratio is a result of an iterative design process. This ratio should be within a range of 0,2 and 0,6 and should be adjustable so that it can be increased as the temperature rises in order to increase denitrification and thus N removal.

E.4 Systems for N removal and simultaneous aerobic sludge stabilization

The sludge age *MSRT* shall be

$$MSRT \geq 25 \cdot 1,072^{\left(12-T_{\min}\right)} \quad [\mathrm{d}] \tag{E.3}$$

The aerobic sludge age *MASRT* shall be

$$MASRT \geq 20 \cdot 1,072^{\left(12-T_{\min}\right)} \quad [\mathrm{d}] \tag{E.4}$$

The surplus sludge is sufficiently stabilized if its BOD_5/COD-ratio does not exceed 0,15 [22]. Extended aeration of the surplus sludge may be required to fulfil this stabilization criterium.

EXAMPLE 1 without extended aeration:

T_{min} = 10 °C: *MASRT* = 23 d;

T = 12 °C: N-removal and thus $V_{\mathrm{Den}}/V_R \approx 0{,}3$ are required. *MASRT* = 20 d. *MSRT* = 20 d / $(1 - V_{\mathrm{Den}}/V_R)$ = 28,5 d.

EXAMPLE 2 with 10 d extended aeration:

T_{min} = 10 °C: *MASRT* = 23 d – 10 d = 13 d.

T = 12 °C: N-removal and thus $V_{\mathrm{Den}}/V_R \approx 0{,}3$ are required. *MASRT* = 10 d. *MSRT* = 10 d / $(1 - V_{\mathrm{Den}}/V_R)$ = 14,3 d.

Because the TSS concentration in the extended aeration tank is about double the concentration in the main reactor, the total volume of the system with extended aeration is only about (14,3 + 10/2)/28,5 = 68 %. However, due to the higher TSS concentration, the oxygen transfer efficiency in the extended aeration tank is lower than that in the main reactor.

Annex F
(informative)

Surplus sludge production

The COD of the biomass ($l_{COD,BM}$) in the surplus sludge can be calculated with the following equation:

$$l_{\mathrm{COD,BM}} = \left(l_{\mathrm{COD,deg.in}} \cdot Y + l_{\mathrm{COD,Dos}} \cdot Y_{\mathrm{COD,Dos}}\right) / \left(1 + b \cdot MSRT \cdot f_T\right) \quad [\mathrm{g/(P \cdot d)}] \tag{F.1}$$

where

Y	is the dimensionless yield coefficient of the COD in the wastewater; it is typically 0,67 kg/kg;
$Y_{COD,Dos}$	is the dimensionless yield coefficient of added carbon in kg/kg; this yield coefficient depends on the type of carbon source (see Table D.1);
$C_{COD,Dos}$	is the COD added as carbon source in kg;
b	is the degradation coefficient in d^{-1} at 15 °C which is dependent on the sludge age MSRT; it is recommended to use the formula: $b = (0{,}065 + 0{,}19 \cdot e^{-(MSRT/20)})$ [d^{-1}] whereby 0,065 d^{-1} is the hydrolysis rate when no substrate is available;
$MSRT$	is the sludge age, i.e. the mean solids retention time in d;
f_T	is the dimensionless temperature factor $f_T = 1{,}072^{(T - 15\ °C)}$;
T	is the wastewater temperature in °C.

A portion of the generated biomass is inert. The specific COD load of the inert biomass ($l_{COD,BM,inert}$) is:

$$l_{\mathrm{COD,BM,inert}} = 0{,}2 \cdot l_{\mathrm{COD,BM}} \cdot MSRT \cdot \mathrm{b} \cdot f_T \quad [\mathrm{g/(P \cdot d)}] \tag{F.2}$$

The specific surplus sludge production ($l_{SSP,C}$) resulting from carbon removal is:

$$l_{SSP,C} = l_{\mathrm{COD,inert,in}} / 1{,}33 + l_{\mathrm{COD,BM}} / 1{,}31 + l_{\mathrm{TSS,inorg,in}} \quad [\mathrm{g/(P \cdot d)}] \tag{F.3}$$

Where phosphorus (P) is chemically precipitated with iron or aluminium salts, additional surplus sludge $l_{P,Prec}$ is generated. The specific mass of P to be precipitated is:

$$l_{P,\mathrm{prec}} = l_{P,\mathrm{in}} - l_{P,\mathrm{out}} - l_{P,\mathrm{BM}} - l_{P,\mathrm{BioP}} \quad [\mathrm{g/(P \cdot d)}] \tag{F.4}$$

where

$l_{P,in}$	is P in the influent (typically 1,6 g/(P·d) to 1,8 g/(P·d)), depending on the wastewater characteristics and the primary treatment;
$l_{P,out}$	is P in the effluent which should be about 65 % of the consent standard;
$l_{P,BM}$	is P in the biomass, about $0{,}005 \cdot l_{COD,in}$;
$l_{P,BioP}$	is additional P in the biomass due to enhanced biological P removal. Under normal circumstances it is about $0{,}006 \cdot l_{COD,in}$. With lower temperatures than 15 °C the value of $l_{P,BioP}$ becomes smaller.

It may be assumed that an addition of 1,5 mol of metal salt is required to remove 1 mol of P. This leads to the addition of 2,7 kg Fe, or 1,3 kg Al, per kg of P.

The additional surplus sludge production $l_{SSP,P}$ resulting from P removal is:

$$l_{\mathrm{SSP,P}} = 3 \cdot l_{P,\mathrm{BioP}} + 6,8 \cdot Xl_{P,\mathrm{Prec,Fe}} + 5,3 \cdot l_{P,\mathrm{Prec,Al}} \quad [\mathrm{g/(P \cdot d)}] \qquad (F.5)$$

where

$l_{P,prec,Fe}$ is amount of P precipitated with iron salts;

$l_{P,prec,Al}$ is amount of P precipitated with aluminium salts.

The entire surplus sludge production l_{SSP} is:

$$l_{SSP} = l_{SSP,C} + l_{SSP,P} \quad [\mathrm{g/(P \cdot d)}] \qquad (F.6)$$

Table F.1 provides an estimate for sludge yields relative to BOD_5.

Table F.1 — Typical sludge yields [Source: United Utilities]

Feed	**Required treatment**	**Sludge yield** kg DS/kg BOD_5
Primary treated wastewater	only BOD removal	1,0
Primary treated wastewater	N removal	0,75
Only preliminary and no primary treatment	N removal	1,1

Additional sludge yields shall be taken into account from:

- Return flows from sludge thickening and dewatering;
- Industrial influents with a high particulate COD fraction;
- Chemical phosphorus precipitation.

Annex G
(informative)

Denitrification capacity

The denitrification capacity ($l_{NO3,Den}$) of the system can be calculated with the following balance:

$$l_{\mathrm{NO3},Den} = l_{N,\mathrm{in}} - l_{\mathrm{orgN,out}} - l_{\mathrm{NH4,out}} - l_{\mathrm{NO3,out}} - l_{\mathrm{orgN,BM}} - l_{\mathrm{orgN,inert}} \quad [\mathrm{g/(P{\cdot}d)}] \tag{G.1}$$

where

$l_{N,in}$	is total nitrogen in the inflow (usually 11 g/(P·d) for municipal wastewater);
$l_{orgN,out}$	is organic nitrogen in the effluent (usually 0,4 g/(P·d));
$l_{NH4,out}$	is dissolved ammonium in the effluent (usually 0 g/(P·d));
$l_{NO3,out}$	is dissolved nitrate in the effluent (usually around 2 g/(P·d));
$l_{orgN,BM}$	is organically bound nitrogen in the biomass and usually 0,07 · $l_{COD,BM}$;
$l_{orgN,inert}$	is particulate inert organic nitrogen and usually 0,03 · ($l_{COD,inert,BM} + l_{COD,inert,in}$).

Annex H
(informative)

Oxygen Consumption

The specific oxygen consumption $OUR_{C,spec}$ for carbon removal is:

$$OUR_{C,spec} = l_{\mathrm{COD,deg,in}} + l_{\mathrm{COD,Dos}} - l_{\mathrm{COD,BM}} - l_{\mathrm{COD,BM,inert}} \quad [\mathrm{g/(P \cdot d)}] \tag{H.1}$$

The following specific oxygen consumption $OUR_{C,redeg,PreD,spec}$ occurs in pre-denitrification (PreD) reactors:

$$OUR_{C,redeg,PreD,spec} = l_{COD,redeg,in} \cdot \left(1 - Y\right) + l_{COD,Dos} \cdot \left(1 - Y_{CSB,Dos}\right) \quad [\mathrm{g/(P \cdot d)}] \tag{H.2}$$

The following specific oxygen consumption $OUR_{C,redeg,intD,spec}$ occurs during intermittent denitrification (intD) if the external Carbon source is dosed during periods of no aearation or during post denitrification (postD):

$$OUR_{C,redec,intD,spec} = l_{COD,Dos} \cdot \left(1 - Y_{CSB,Dos}\right) \quad [\mathrm{g/(P \cdot d)}] \tag{H.3}$$

The ratio V_{Den}/V_R or the ratio of denitrification time to cycle time for intermittent denitrification (t_{Den}/t_{cy}) is initially assumed. An iterative calculation is required (see Annex I).

The specific oxygen consumption of denitrification ($OUR_{C,Den,spec}$), the oxygen taken from nitrate, depends on the system configuration. It is

- for pre-denitrification PreD:

$$OUR_{C,PreD,spec} = 0{,}75 \cdot \left[OUR_{C,redeg,PreD,spec} + \left(OUR_C - OUR_{C,redeg,PreD,spec} \right) \cdot \left(V_{Den} / V_R \right)^{0{,}68} \right] \quad [\mathrm{g/(P \cdot d)}] \tag{H.4}$$

- for intermittent denitrification intD:

$$OUR_{C,IntD.spec} = 0{,}75 \cdot \left[OUR_{C,redeg,IntD,spec} + \left(OUR_C - OUR_{C,redeg,IntD,spec} \right) \cdot \left(t_{Den} / t_{cy} \right) \right] \quad [\mathrm{g/(P \cdot d)}] \tag{H.5}$$

- for simultaneous denitrification (simD) without a previous anaerobic reactor:

$$OUR_{C,simD,spec} = 0{,}75 \cdot OUR_C \cdot \left(V_{Den} / V_R \right) \quad [\mathrm{g/(P \cdot d)}] \tag{H.6}$$

The specific oxygen consumption for nitrification is:

$$OUR_{N,spec} = 4{,}3 \cdot \left(l_{NO3,Den} - l_{NO3,in} + l_{NO3,out} \right) \quad [\mathrm{g/(P \cdot d)}] \tag{H.7}$$

The oxygen consumption is reduced by denitrification:

$$OUR_{Den,spec} = 2{,}86 \cdot l_{NO3,Den} \quad [\mathrm{g/(P \cdot d)} \tag{H.8}$$

The overall oxygen consumption per hour of a system is:

$$OUR = OUR_C + OUR_N - OUR_{Den} = \left(OUR_{C,spec} + OUR_{N,spec} - OUR_{Den,spec} \right) \cdot \frac{PT}{24} \quad [\mathrm{kg/h}] \tag{H.9}$$

Variations of loads shall be taken into account. There might be seasonal variations, e.g. in resort areas or during wine harvesting campaigns. The highest load shall be determined. And there are hourly variations with the highest load usually occurring early in the morning. The maximum hourly oxygen consumption is:

$$OUR_{h,max} = \left[f_C \cdot \left(OUR_{C,max} - OUR_{C,Den,max} \right) + f_N \cdot OUR_{N,\max} \right] / 24 \quad \text{[kg/h]} \qquad \text{(H.10)}$$

The carbon and nitrogen surge factors f_C and f_N depend on sludge age and plant size and can be taken from Table H.1. The maximum of $OUR_{h,max}$ shall be determined both for f_N while $f_c = 1$ and for f_C while $f_N = 1$, i.e. it is assumed that the carbon and nitrogen peaks do not occur simultaneously. The nitrogen peak is usually dominant.

Table H.1 — Carbon and nitrogen surge factors f_C and f_N [20]

	Sludge age d							
	2	4	6	8	10	15	25	35
f_C	1,4	1,3	1,25	1,2	1,2	1,15	1,1	1,05
f_N[a] for < 2 400 kg COD/d	No nitrogen removal				2,4	2,0	1,5	1,1
f_N[a] for > 12 000 kg COD/d	No nitrogen removal				1,8	1,5	1,3	1,1

[a] Where the hourly N loads are measured, the determined peak factors f_N shall be used.

The hourly oxygen consumption shall be determined for a variety of scenarios, e.g. as a maximum at maximum temperature (with N-removal), as a minimum at minimum temperature (perhaps without nitrification) and as a medium at average temperature (with N-removal, e.g. at 15 °C). Seasonal variations of the loads shall also be taken into consideration, e.g. in holiday resorts. In addition, present and future loads shall be taken into account. The power consumption of the system over a time period shall be determined dependent on its expected duration.

Annex I
(informative)

Iterative calculation of the volumetric ratio of denitrification reactors (V_{Den}/V_R)

This calculation needs to be iterative. The following steps shall be taken:

1. An initial value for V_{Den}/V_R ot t_{Den}/t_{cy} has been assumed in Annex H;
2. The sludge age *MSRT* is calculated with Formula (E.2) in Annex E;
3. The specific surplus sludge production SSP_{spec} is calculated with Formula (F.6) in Annex F;
4. The denitrification capacity $l_{NO3,Den}$ is calculated with Formula (G.1) in Annex G;
5. $OUR_{C,spec}$ is calculated with Formula (H.1) in Annex H;
6. $OUR_{C,redeg,xDen}$ is calculated with Formula (H.2) or Formula (H.3) in Annex H;
7. $OUR_{C,xDen}$ is calculated with Formula (H.4), (H.5) or Formula (H.6) in Annex H;
8. The ratio $x = OUR_{C,xDen,spec} / (2,86 \cdot l_{NO3,Den})$ is calculated;
9. If $x > 1$, V_{Den}/V_R or t_{Den}/t_{cy} or the addition of a carbon source (where used) may be reduced;
10. If $x < 1$, V_{Den}/V_R or t_{Den}/t_{cy} or the addition of a carbon source (where used) shall be increased;
11. If $x \approx 1$, V_{Den}/V_R or t_{Den}/t_{cy} is correct, the iteration is finished.

Annex J
(informative)

Reactor volume (V_R)

The reactor volume (V_R) is the product of the sludge age ($MSRT$) and the specific surplus sludge production (SSP_{spec}), divided by the suspended solids concentration in the reactor (MLSS concentration or $C_{TSS,R}$) and multiplied with the total population (PT).

$$V_R = SSP_{spec} \cdot MSRT \cdot PT \;/\; C_{TSS,R} \quad [m^3] \qquad (J.1)$$

The MLSS concentration in the reactor ($C_{TSS,R}$) depends on the sludge volume index (SVI), i.e. the thickening characteristics of the sludge, and the design of the final clarifier (see Annex Q).

Annex K
(informative)

Internal recirculation ratio (IRR)

Assuming that the nitrate concentration in the influent is negligible, the internal recirculation ratio can be calculated with Formula (K.1):

$$IRR = l_{NO3,Den} \; / \; l_{NO3,out} \qquad [-] \qquad \text{(K.1)}$$

Some additional nitrate is recirculated with the return sludge. For this reason, *IRR* can be somewhat lower.

The flow of internal recirculation pumps should be controlled via the nitrate concentration in the effluent.

Annex L
(informative)

Alkalinity

The alkalinity is reduced by nitrification and phosphorous precipitation with acidic metal salts, but somewhat increased by denitrification. This can be a problem where the water is soft. The alkalinity $c_{alk,out}$ should not drop below a value of 1,5 mol/m³.

$$c_{\mathrm{alk,out}} = c_{\mathrm{alk,in}} - 0{,}07 \cdot \left(C_{\mathrm{NH4,in}} - C_{\mathrm{NH4,out}} + C_{\mathrm{NO3,out}} - C_{\mathrm{NO3,in}}\right) + 0{,}06 \cdot C_{Fe3} + 0{,}04 \cdot C_{\mathrm{Fe2}} + 0{,}11 \cdot C_{\mathrm{Al3}} - 0{,}03 \cdot C_{\mathrm{P,prec}} \quad [\mathrm{mol/m^3}] \tag{L.1}$$

where

$c_{alk,in}$	is the alkalinity of the inflow in mol/m³;
$c_{alk,out}$	is the alkalinity of the effluent in mol/m³;
$C_{NH4,in}$	is the ammonium concentration of the inflow in g/m³;
$C_{NH4,out}$	is the ammonium concentration of the effluent in g/m³;
$C_{NO3,in}$	is the nitrate concentration of the inflow in g/m³;
$C_{NO3,out}$	is the nitrate concentration of the effluent in g/m³;
C_{Fe3}	is the concentration of added Fe^{3+} in g/m³;
C_{Fe2}	is the concentration of added Fe^{2+} in g/m³;
C_{AL3}	is the concentration of added Al^{3+} in g/m³;
$C_{P,prec}$	is the amount of P removed be precipitation in g/m³.

The alkalinity of the effluent $c_{alk,out}$ should not drop below a value of 1,5 mol/m³ and the pH-Value of the effluent should be above 6,5. Otherwise caustic chemicals (e.g. lime) might need to be added.

The more efficient a diffuser system is (especially in deep reactors), the lower can the pH-value drop, because less acidic CO_2 is stripped out.

EXAMPLE Assuming a diffuser system with a specific oxygen transfer rate of 20 g O_2/(Nm³·m) in clean water and 14 g O_2/(Nm³·m) in mixed liquor (assuming an α-factor of 0,7), and an immersion depth of 5 m, the oxygen transfer ratio is.

14 g O_2/(Nm³ · m) · 5 m / 300 g O_2/Nm³ = 23 %

where

300 g O_2/Nm³ is the oxygen concentration of air at norm conditions. In this case an $c_{alk,out}$ of minimum 2,5 mol/m³ is required (see Table L.1).

Table L.1 — pH values in aerated reactors depending on the oxygen transfer efficiency of aeration systems under operational conditions [Source: Teichgräber 1991]

$S_{alk,out}$ [mol/m³]	pH-value in aeration reactors depending on the oxygen transfer rate				
	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

Annex M
(informative)

Aerobic selectors

Aerobic selectors shall provide a retention time for the sum of the average dry weather inflow and the return sludge flow of 30 min to 40 min.

They should be comprised of a two-unit cascade. Their volumetric load should be about 20 kg COD per m^3 and day.

Their required clean water oxygen transfer rate per m^3 is about αOC = 0,17 kg $O_2/(m^3 \cdot h)$.

Annex N
(informative)

Design based on F/M-ratio

Table N.1 and Table N.2 show design values in the temperature range 10°C to 12°C for the F/M-ratio (based on the BOD_5 load) and the hydraulic retention time (*HRT*) dependent of the required ammonia concentration in the effluent. N-removal, i.e. denitrification, is not required.

Table N.1 — Maximum F/M-ratio and minimum HRT for raw wastewater [Source: Yorkshire Water]

Effluent NH_4-N concentration mg/l	**Max. F/M-ratio** kg BOD_5/(kg MLSS · d)	**Min. HRT** h
1	0,05	6,5
3	0,06	6
5	0,07	5,5
10	0,1	5

Table N.2 — Maximum. F/M-ratio and minimum. HRT for pre-clarified wastewater [Source: Yorkshire Water]

Consent standard **Effluent NH_4-N** mg/l	**Max. F/M-ratio** kg BOD_5/(kg MLSS·d)	**Min. HRT** h
1	0,07	5
2	0,08	5
3	0,09	4,5
5	0,1	4
7	0,11	4
10	0,125	3,5
No nitrification, only BOD_5-removal	0,3	2,5

Annex O
(informative)

Sludge volume index (SVI)

Table O.1 — Typical values of the sludge volume index SVI [DWA-A 131]

Process objective	*SVI* ml/g	
	Low (favourable industrial influence)	High (unfavourable industrial influence)
Only carbon removal	100 to 150	120 to 180
Nitrification with or without denitrification	100 to 150	120 to 180
Aerobic sludge stabilization	75 to 120	100 to 150

Annex P
(informative)

Solids concentration of the return sludge ($C_{TSS,RS}$)

The suspended solids concentration at the bottom of the final clarifier ($C_{TSS,B}$) depends on the sludge characteristics (the sludge volume index SVI in ml/g) and the thickening time (t_{th} in hours) in the clarifier:

$$C_{\mathrm{TSS},B} = 1\,000 \cdot t_{\mathrm{th}}{}^{0{,}33} \;/\; SVI \quad [\mathrm{kg/m^3}] \qquad \text{(P.1)}$$

where

t_{th} should be about 2 h.

The suspended solids concentration of the return sludge ($C_{TSS,RS}$) is:

$$C_{\mathrm{TSS,RS}} = f_{SE} \cdot C_{\mathrm{TSS},B} \quad [\mathrm{kg/m^3}] \qquad \text{(P.2)}$$

where the scraper's effectivity factor f_{SE} is:

f_{SE} = 0,7 − 0,8 for shield and bar scrapers,

f_{SE} = 0,5 − 0,7 for suction scrapers, and

f_{SE} = 1 for small and deep clarifiers with predominantly vertical flow and no scraper.

Annex Q
(informative)

Return sludge flow (Q_{RS}) and total suspended solids concentration in the biological reactor ($C_{TSS,R}$)

Q.1 General

The return sludge flow rate Q_{RS} is the product of the maximum design flow Q_{max} and the return sludge ratio RSR:

$$Q_{RS} = Q_{max} \cdot RSR \quad [m^3/h] \tag{Q.1}$$

The suspended solids concentration in the biological reactor ($C_{TSS,R}$) is:

$$C_{TSS,R} = RSR \cdot C_{TSS,RS} \; / \left(1 + RSR\right) \quad [kg/m^3] \tag{Q.2}$$

The *RSR* should be controlled by measuring the inflow and adjusting variable speed pumps such that a ratio of about 50 % is maintained.

NOTE It is assumed that the maximum wet weather flow does not exceed 200 % of the dry weather flow. National or local regulations can require a different percentage and therewith a different design.

Q.2 Surface flow rate (q_A) and surface sludge flow rate (q_{SV})

The surface flow rate q_A is:

$$q_A = q_{SV} \; / \; \left(C_{TSS,R} \cdot SVI\right) \quad [m/h] \tag{Q.3}$$

where

q_{SA} shall not exceed 0,5 m/h for clarifiers with predominantly horizontal flow, and 0,65 m/h for clarifiers with predominantly vertical flow;

q_A shall not exceed 1,6 m/h for clarifiers with predominantly horizontal flow, and 2,0 m/h for clarifiers with predominantly vertical flow;

SVI is the sludge volume index in ml/g indicating the volume a mass of sludge assumes after sedimentation.

Predominantly horizontal flow clarifiers have a ratio of the depth of the inflow (below the water surface) to the horizontal distance between in- and outflow of maximum 1 : 3.

Predominantly vertical flow clarifiers have a ratio of the depth of the inflow (below the water surface) to the horizontal distance between in- and outflow of minimum 1 : 2.

Table Q.1 provides information about the design of intermediate clarifiers, depending on the Q_{ver}/Q_{hor} ratio. The ratio of vertical to horizontal flow (Q_{ver}/Q_{hor} ratio) is the ratio of the vertical distance from the inflow to the overflow (= $h_1 + h_2$) to the horizontal distance from the inflow to the overflow.

Table Q.1 — Clarifier design depending on the ratio of vertical to horizontal flow [DWA-A 131]

Parameter	**Unit**	**Design values**						
Q_{ver}/Q_{hor} ratio	—	≥ 0,33	≥ 0,36	≥ 0,39	≥ 0,42	≥ 0,44	≥ 0,47	≥ 0,5
q_{SV}	$m^3/(m^2 \cdot h)$	≤ 0,5	≤ 0,525	≤ 0,55	≤ 0,575	≤ 0,6	≤ 0,625	≤ 0,65
q_A	m/h	≤ 1,6	≤ 1,65	≤ 1,75	≤ 1,8	≤ 1,85	≤ 1,9	≤ 2,0
RSR	—	≤ 0,75	≤ 0,8	≤ 0,85	≤ 0,9	≤ 0,9	≤ 0,95	≤ 1

Annex R
(informative)

Surface area (A_{Cla}) of final clarifiers

The required surface area is:

$$A_{Cla} = Q_{\max} \;/\; q_A \quad [\mathrm{m}^2] \tag{R.1}$$

where

Q_{max} is design flow at wet weather conditions in m³/h and

q_A is the surface flow rate in m/s

For predominantly vertical flow final clarifiers, which have a conical shape, A_{Cla} is the area at a level, which is half way between the inflow level and the water surface.

Annex S
(informative)

Depth (h_{Cla}) of final clarifiers

The depth h_{Cla} at 2/3rd of the horizontal distance from the inflow to the overflow is calculated.

There are three distinct zones with heights h_x in a clarifier:

- h_1: The clear water zone underneath the surface;
- h_2: the transition and storage zone;
- h_3: the gravity thickening and sludge removal zone.

$$h_1 = 0{,}5\text{ m} \quad [\text{m}] \tag{S.1}$$

$$h_2 = q_A \cdot (1 + RSR) \cdot \left[500 \,/ \left(1\,000 - C_{\text{TSS},R} \cdot SVI\right) + C_{\text{TSS},R} \cdot SVI \,/ 1\,100\right] \quad [\text{m}] \tag{S.2}$$

$$h_3 = q_A \cdot C_{\text{TSS},R} \cdot (1 - RSR) \,/\, C_{\text{TSS},B} \quad [\text{m}] \tag{S.3}$$

$$h_{Cla} = h_1 + h_2 + h_3 \quad [\text{m}] \tag{S.4}$$

h_{Cla} shall be minimum 3 m. The water depth of circular clarifiers at the side wall shall be minimum 2,5 m.

For conical Dortmund clarifiers with predominantly vertical flow the volumes of the zones need to be calculated taking account of the clarifier geometry. The volumes of the zones shall be equal to $h_x \cdot D_{Cla}$, where D_{Cla} is the diameter of the clarifier. This can lead to an increase of D_{Cla} and h_{Cla}.

The lower edge of the inflow structure shall be minimum 0,3 m above the upper level of the gravity thickening and sludge removal zone.

Annex T
(informative)

Scraper Design

Sludge scraping Table T.1 provides information about scraper design.

Table T.1 — Typical design of scrapers [Source: DWA-A 131]

Parameter	Symbol	Unit	Circular clarifier	Rectangular clarifier	
			Shield scraper	**Shield scraper**	**Chain scraper**
Shield or bar height	h_S	m	0,3 to 0,6	0,3 to 0,8	0,15 to 0,3
Shield velocity	v_S	m/h	72 to 144	Max. 108	36 to 108
Return velocity	v_R	m/h	—	Max. 324	—
Scraping factor	f_S	—	0,67	≤ 1,1	≤ 1,25

The scraping factor f_S is the ratio of the sludge volume actually moved and the theoretically (geometrically) moved sludge volume. This factor may be above 1,0 in rectangular clarifiers because a sludge layer above the upper edge of shields or bars can be moved along with the sludge below.

The shield velocity v_S is its velocity at the side wall.

Circular clarifiers

The scraped sludge flow Q_{Scr} is:

$$Q_{Scr} = h_{Scr} \cdot a \cdot v_{Scr} \cdot D_{Cla} \;/\; \left(4 \cdot f_{Scr}\right) \quad [\mathrm{m^3/h}] \tag{T.1}$$

where

- a_{Scr} is the number of scraper arms; and
- D_{Cla} is the clarifier diameter in meters.
- Q_{Scr} is the scraped sludge flow in m³/h
- h_{Scr} is the shield or bar height in m
- v_{Scr} is the shield velocity in m/h
- f_{Scr} is the scraping factor

Rectangular clarifiers

Shield scrapers:

The cycle time t_{cy} of shield scrapers is:

$$t_{cy} = L_{Cla} \;/\; v_{Scr} + L_{\mathrm{Cla}} \;/\; v_{ret} + t_{del} \quad [\mathrm{h}] \tag{T.2}$$

where

- L_{Cla} is the traveling length of the scraper in meters (it should not exceed 40 m, and 60 m if additional sludge hoppers are provided at about half length); and

t_{del} is the time in seconds needed for raising and lowering the shield.

v_{Scr} is the shield velocity in m/h

v_{ret} is the return velocity in m/h

The scraper sludge flow Q_S is:

$$Q_S = h_{Scr} \cdot W_{Scr} \cdot L_{Cla} \ / \ \left(f_{Scr} \cdot t_{cy}\right) \quad [m^3/h] \tag{T.3}$$

where

W_{Scr} is the width of the scraper in metres.

Chain scrapers:

The scraper sludge flow Q_{Scr} is:

$$Q_{Scr} = h_{Scr} \cdot W_{Scr} \cdot h_{Scr} \ / \ f_{Scr} \quad [m^3/h] \tag{T.4}$$

The distance from bar to bar should be about 15 times the bar height h_S.

Suction scrapers in circular or rectangular clarifiers:

The return sludge flow Q_{RS} is the pump flow. The return sludge concentration $C_{TSS,RS}$ is lower than the bottom sludge concentration $C_{TSS,B}$ because of some short-circuiting from the transition and storage zone.

The velocity in the suction pipes should be 0,6 m/s to 0,8 m/s. The distance between suction pipes should not exceed 3 m to 4 m. The scraper velocity is about the same as that of shield scrapers. In circular clarifiers the suction capacity shall be proportional to the radius from the centre.

Annex U
(informative)

Return sludge balance

The return sludge flow Q_{RS} is usually higher than the scraper sludge flow Q_{Scr}:

$$Q_{RS} = Q_{Scr} + Q_{SC} \quad [m^3/h] \tag{U.1}$$

where Q_{SC} is a short-cut flow from the transition and storage zone with a concentration similar to that in the biological reactor $C_{SS,R}$.

A check shall be performed to determine whether the chosen factor f_{SE} in Annex P is achieved:

$$f_{SE} = C_{TSS,RS} \ / \ C_{TSS,B} = \left(Q_{Scr} \cdot C_{TSS,B} + Q_{SC} \cdot C_{TSS,R}\right) \ / \ \left(Q_{RS} \cdot C_{TSS,B}\right) \quad [-] \tag{U.2}$$

Where the factor f_{SE} assumed in Annex P is not met, $C_{TSS,R}$ in Formula (Q.2) (in Annex Q) shall be reduced or the clarifier's design shall be improved.

Annex V
(informative)

Influent structures

The inflow into clarifiers shall be within a range of 0,3 m to 0,6 m above the thickening zone h_3. Because the upper level of the thickening is usually higher during high flow and lower during low flow, a height adjustable inflow structure might be useful.

The horizontal inflow velocity shall be below 7 cm/s at the maximum inflow Q_{max}.

Annex W
(informative)

Design of a fine bubble aeration system

Table W.1 shows a design example for future maximum load. The maximum air flow through diffusers usually occurs during summer because of maximum denitrification capacity (V_D/V_R = max.) and minimum aerated reactor volume. The minimum air flow per diffuser occurs during winter when the anoxic denitrification volume is minimal.

The same calculations shall be done with peak factors $f_N = 1$ and $f_C = 1$ (see Annex H) for the maximum (usually during summer), the average (usually during spring and autumn) and the minimum load (usually during winter without denitrification ($V_{Den}/V_R = 0$).

Table W.1 — Design example of a fine bubble diffuser system

Parameter		Equation	Unit	Value
OC_h	Hourly oxygen consumption	Result from Formula (H.10)	kg O_2/h	100
T	Reactor temperature at peak load		°C	18
h_{geo}	Geodetic height of site		m	400
p_{atm}	Atmospheric pressure	$P_{atm} = 101,3 \cdot \left[\frac{\left(288 - 0,0065 \cdot h_{geo}\right)}{288}\right]^{5,255}$	kPa	96,6
V_{aer}	Aerated reactor volume	$V_{aer} = V_R \cdot \left(1 - V_{Den} / V_R\right)$	m³	1 000
h_R	Water depth in reactor	selected	m	4,2
h_{Dif}	Immersion depth	$h_{Dif} \approx h_R - 0,2$ m	m	4,0
f_h	Depth factor	$f_h \approx 1 + h_{Dif} / 30m$ only valid between 3 m and 8 m immersion depth (h_{Dif})	—	1,13
A_R	Surface area of reactor	$A_R = V_R / h_R$	m²	228
α	Ratio of mixed liquor to clean test water oxygen tranfer	Dependent on wastewater quality and $C_{TSS,R}$. To be stated by the system designer.	—	0,65
$C_{Sal,TW}$	Salinity of clean test water		g/l	0,2
$ß_{Sal,TW}$	Salt factor of clean test water	$ß_{Sal,TW} = 1 - 0,01 \cdot C_{Sal,TW}$	—	0,998
$f_{kLa,TW}$	k_La salt factor of clean test water	$f_{kLa,TW} = 1 + 0,08 \cdot C_{Sal,TW}$	—	1,02

Parameter		Equation	Unit	Value
$C_{O_2,Sat,20}$	Oxygen saturation at 20 °C in clean test water		mg/l	9,1
$C_{Sal,ML}$	Salinity of mixed liquor		g/l	2,0
$ß_{Sal,ML}$	Salt factor of mixed liquor	$ß_{\alpha Sal,ML} = 1 - 0,01 \cdot C_{\mathrm{Sal,ML}}$	—	0,98
$f_{kLa,ML}$	k_{L}a salt factor of mixed liquor	$f_{kLa,ML} = 1 + 0,08 \cdot C_{\mathrm{Sal,ML}}$	—	1,16
$C_{O_2,Sat,T}$	*Oxygen saturation at T in clean test water*	$c_{O2,Sat,T} = 2\,234 / (T + 46)^{1,134}$	mg/l	9,46
$C_{O_2,R}$	Oxygen concentration in reactor	process dependent, e.g. 2,0 mg/l for nitrification or 1,5 mgt/l for simultaneous aerobic stabilization	mg/l	2,0
f_{int}	Factor for intermittent aeration	$f_{int} = 1 / (1 + t_{Den} / t_{cy})$	—	1
SOTR	Standard oxygen transfer rate	$(f_{\mathrm{h}} \cdot ß_{\mathrm{Sal,TW}} \cdot C_{O_2,\mathrm{Sat,20}} \cdot f_{\mathrm{kLa,TW}} \cdot OC_{\mathrm{h}} \cdot f_{\mathrm{int}})$ / $[(f_{\mathrm{h}} \cdot ß_{\mathrm{Sal,ML}} \cdot C_{O_2,\mathrm{Sat,T}} \cdot p_{\mathrm{atm}}/1\,013 - C_{O_2,\mathrm{R}}) \cdot \alpha \cdot f_{\mathrm{kLa,ML}} \cdot 1,024^{(T-20)}]$	kg O_2/h	182
SSOTR	Specific standard oxygen transfer rate (T = 20 °C, p = 1 013 hPa, C_{O_2} = 0 mg/l)	Dependent on diffuser type, a_{Dif} and $q_{\mathrm{Air,St,Dif}}$ (see below). An iterative calculation is usually necessary.	g/(Nm³·m)	20
SSOTE	Specific standard oxygen transfer efficiency (T = 20 °C, p = 1 013 hPa, C_{O_2} = 0 mg/l)	*SSOTE = SSOTR*/3	%/m	6,6
$Q_{Air,St}$	Standard air flow	$Q_{\mathrm{Air,St}} = 1\,000 \cdot SOTR/(SSOTR \cdot h_{\mathrm{Dif}})$	Nm³/h	2 275
	Type of diffuser	Membrane disk diffuser	—	—
$q_{Air,St,Dif,max}$	Max. standard air flow per diffuser	Dependent on diffuser type and immersion depth	Nm³/h	6
$n_{Dif,min}$	Minimum number of diffusers	$N_{\mathrm{Dif,min}} = Q_{\mathrm{Air,,St}}/q_{\mathrm{Air,St,Dif,max}}$	—	379
n_{Dif}	Number of diffusers	selected	—	400
$q_{Air,St,Dif}$	Standard air flow per diffuser	$q_{\mathrm{Air,tS,Dif}} = Q_{\mathrm{Air,St}}/n_{\mathrm{Dif}}$	Nm³/h	5,7
$A_{Dif,eff}$	Effective area per diffuser	Dependent on diffuser type	m²	0,08
F_{Dif}	Diffuser ensity	$F_{\mathrm{Dif}} = 100 \cdot n_{\mathrm{Dif}} \cdot A_{\mathrm{Dif,eff}} / A_{\mathrm{R}}$	%	14,0
A_{Dif}	Floor area per diffuser	$A_{\mathrm{Dif}} = A_{\mathrm{R}}/n_{\mathrm{Dif,sel}}$	m²	0,57
$q_{Air,A}$	Air flow rate per floor area	$q_{\mathrm{Air,A}} = Q_{\mathrm{Air,St}}/A_{\mathrm{R}}$	Nm³/(m²·h)	10

Parameter		Equation	Unit	Value
p_{im}	Immersion pressure	$p_{\mathrm{im}} = p_{\mathrm{atm}} + 98{,}1 \cdot h_{\mathrm{Dif}}$	hPa	1 358
P_R	Power input per reactor volume	$P_{\mathrm{R}} = 3{,}5 \cdot Q_{\mathrm{Air,St}} \cdot 1\,013 \cdot [1 - (p_{\mathrm{atm}}/p_{\mathrm{im}})^{0{,}29}] / (36 \cdot \mathrm{V_R})$	W/m³	21,1
Δp_{Dif}	Pressure loss in diffusers	Dependent on diffuser type and air flow per diffuser	hPa	30
Δp_{PL}	Pressure loss in air pipeline	Dependent on pipeline design, point losses and air flow rate	hPa	20
Δp_{Bl}	Net pressure increase by blower	$\Delta p_{\mathrm{Bl}} = \Delta p_{\mathrm{Dif}} + \Delta p_{\mathrm{PL}} + p_{\mathrm{im}} - p_{\mathrm{atm}}$	hPa	442
T_{atm}	Max. atmospheric temperature		K °C	303 30
T_{out}	Air temperature after blower	$T_{\mathrm{out}} = T_{\mathrm{atm}} \cdot [(p_{\mathrm{atm}} + \Delta p_{\mathrm{Bl}})/p_{\mathrm{atm}}]^{0{,}29}$	K °C	338 65
P_{Bl}	Gross power consumption of blower(s)	Dependent on type of blower, p_{atm}, $T_{\mathrm{Atm,max}}$, air humidity	kW	45
$SOTE$	Standard oxygen transfer efficiency	$SOTE = SOTR/P_{Bl}$	kg O_2/kWh	4,04
OTE	Oxygen transfer efficiency at operational conditions	$OTE = OC_h/P_{Bl}$	—	2,22

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