Examination CT3420 – Sanitary engineering

Date	:	9 April 2010
Time	:	9.00 - 12.00

This exam consists of 3 parts. Each part contains questions with a total of 100 points. The end score is between 1 and 10, proportional to the total number of points. A minimum of 159 points (53%) is required to pass the exam.

With every question you have to show that you able to determine the influence of process parameters so you can optimize the design and the operation of the treatment processes.

If there is something unclear in the questions please inform the supervisor.

An A4 with own remarks is not allowed. Added to this exam are the most important equations.

Always give a motivation to your answer and ask yourself if the answer is complete and if the treatment process can be constructed in field practice.

Use a separate answer page for every part. Write your name and study number clearly on every answer page.

Students are allowed to give their answers in the Dutch laguage.

Formula sheet CT3420 - Drinking water

Table 1 – Atomic mass of the most important elements in water chemistry.

Element	Atomic mass	Element	Atomic mass
H	1	S	32
C	12	Cl	35,5
N	14	K	39
O	16	Ca	40
F	19	Mn	55
Na	23	Fe	56
Mg	24	As	75
Al	27	Pb	207
P	31		

Table 2 - Dynamic and kinematic viscosity as a function of temperature.

Temperature [°C]	Dynamic viscosity $[10^{-3}\ Pa \cdot s]$	Kinematic viscosity $[10^{-6}\ m^2/s]$
0	1.79	1.79
5	1.52	1.52
10	1.31	1.31
15	1.15	1.15
20	1.01	1.01
25	0.90	0.90
30	0.80	0.80

Equilibrium reactions calcium carbonate :

$CO_2 + 2H_2O$	<-->	$H_3O^+ + HCO_3^-$	$K_1 = 3.44 \cdot 10^{-7}$	$pK_1 = 6.46$
$HCO_3^- + H_2O$	<-->	$H_3O^+ + CO_3^{2-}$	$K_2 = 3.25 \cdot 10^{-11}$	$pK_2 = 10.49$
$CaCO_3$	<-->	$Ca^{2+} + CO_3^{2-}$	$K_s = 3.80.10^{-9}$	$pK_s = 8.36$
$CaCO_3 + CO_2 + H_2O$	<-->	$Ca^{2+} + 2HCO_3^-$	$K_a = 4.11 \cdot 10^{-5}$	$pK_a = 4.33$

(K values at T = 10°C)

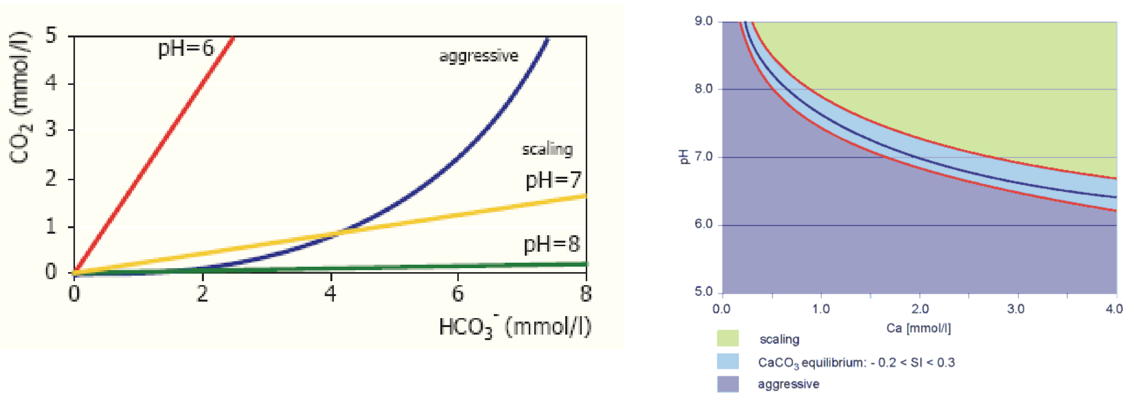


Figure 1. Tillmans-curves (on the left the relation CO_2-HCO_3^-, on the right the relation Ca-pH)

Gases in water:	
Gas exchange:	$\frac{dc}{dt} = k_2 \cdot (c_s - c), \quad \frac{c_s - c}{c_s - c_0} = e^{-k_2 \cdot t}$
General gas law:	$c_g = p_a/(RT)$ $R = 8,3143\ J\ mol^{-1}\ K^{-1}$
Henry's law:	$c_s = k_d.c_g$ (mol/m^3)

Table 3 - k_D-values for different gases as a function of temperature.

k_D	0°C	10°C	20°C
Nitrogen	0.023	0.019	0.016
Oxygen	0.049	0.041	0.033
Methane	0.055	0.043	0.034
Carbon dioxide	1.710	1.230	0.942
Hydrogen sulphide	4.690	3.650	2.870
Tetrachloroethene	-	3.380	1.880
Trichloroethene	-	4.100	2.390
Chloroform	-	9.620	5.070

Table 4 – Composition of air in volume% at 10 °C and under atmospheric pressure (101325 Pa).

Gas	Composition [volume percentage]
Nitrogen	78.084
Oxygen	20.948
Argon	0.934
Carbon dioxide	0.034
Methane	0.0001

Pipelines:	
Friction losses (Darcy-Weisbach):	$\Delta H_w = \lambda \cdot \frac{L}{D} \cdot \frac{v^2}{2 \cdot g}$ $\lambda = 0.02$
Total local losses:	$\Delta H_v = \sum \xi \cdot \left(\frac{v^2}{2 \cdot g} \right)$
Total cost pipelines:	$K_{total} = 19.2 \cdot 16.7 \cdot Q^3 \cdot D^{-5} \cdot L + 500 \cdot D \cdot L$ (*50 years*)

Relevant formulas aeration / degassing:

$$K_1 = 1 - e^{(-k_2 \cdot t)} \qquad K_2 = \frac{1}{1 + \frac{1}{k_2 \cdot t}}$$

$$K_3 = \frac{1 - e^{\left(-k_2 \cdot t \left(1 + \frac{k_d}{RQ}\right)\right)}}{1 + \frac{k_d}{RQ}}$$

$$K_4 = \frac{1 - e^{\left(-k_2 \cdot t \left(1 - \frac{k_d}{RQ}\right)\right)}}{1 - \frac{k_d}{RQ} \cdot e^{\left(-k_2 \cdot t \left(1 - \frac{k_d}{RQ}\right)\right)}}$$

$$K_5 = \frac{1}{1 + \frac{1}{k_2 \cdot t} + \frac{k_d}{RQ}}$$

Relevant formulas filtration:

$$I_0 = \frac{H_0}{L} = 180 \cdot \frac{\nu}{g} \cdot \frac{(1 - p_0)^2}{p_0^{\ 3}} \cdot \frac{v}{d_0^{\ 2}}$$

$$H = 130 \cdot \frac{\nu^{0.8}}{g} \cdot \frac{(1 - p_e)^{1.8}}{p_e^{\ 3}} \cdot \frac{v^{1.2}}{d^{1.8}} \cdot L_e$$

Relevant formules sedimentation:

$$v_s = \frac{1}{18} \cdot \frac{g}{\nu} \cdot \frac{\rho_s - \rho_w}{\rho_w} \cdot d^2$$

$$Re = \frac{v_o \cdot R}{\nu}$$

$$c_p = \frac{v_o^{\ 2}}{g \cdot R}$$

Formule sheet CT3420 - Urban drainage

Weir formula:

$$Q = mBh^{\frac{3}{2}}$$

Where:

Q discharge in m^3/s
m weir coefficient in $m^{0,5}/s$
h heigth of overflow jet in m

Local losses:

$$\Delta H = \xi \frac{Q|Q|}{2gA^2}$$

Where:

ΔH energy loss in m
ξ Loss coefficient (dimensionless)
Q discharge in m^3/s
A wet cross-section in m^2
g gravitation in m/s^2

Friction losses in a conduit:

$$\Delta H = \frac{Q|Q|L}{C^2 R_h A^2}$$

Where:

ΔH energy loss in m
C Chezy coefficient in $m^{0,5}/s$
Q discharge in m^3/s
L length of conduit in m
R_h hydraulic radius in m
A wet cross-section in m^2

The hydraulic radius R_h is defined as:

$$R_h = \frac{A}{P}$$

where:

A wet cross-section in m^2
P wetted perimeter in m

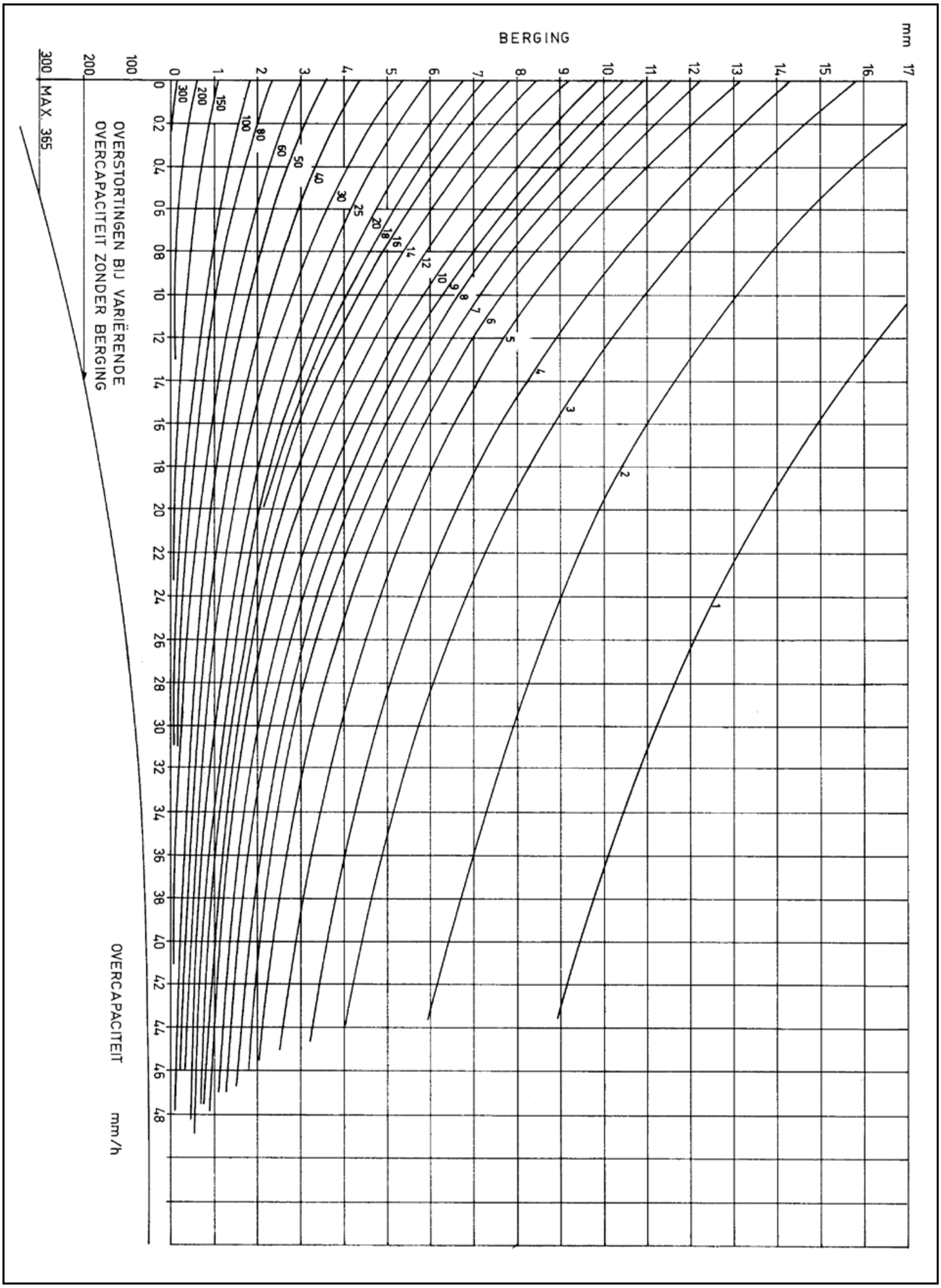
The Chezy coefficient is defined as:

$$C = 18^{10}\log\left[\frac{12R_h}{k_n}\right]$$

where:

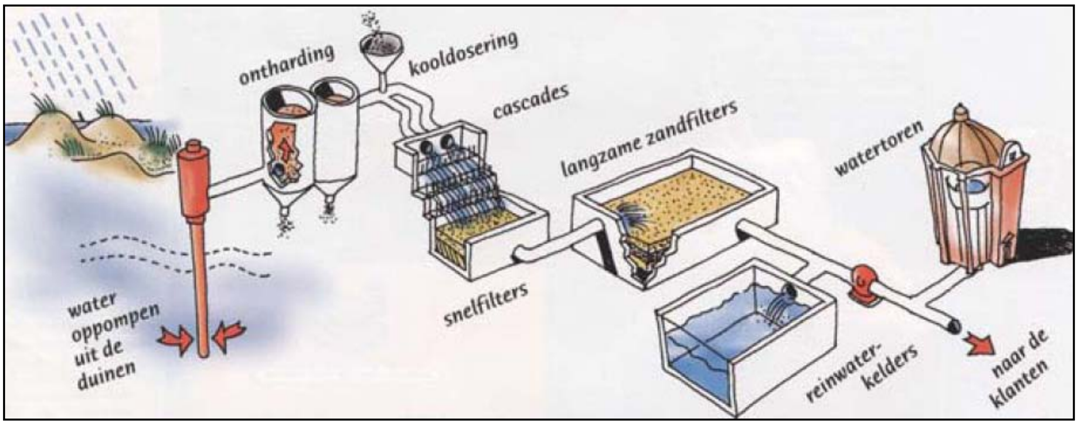
C Chezy coefficient in $m^{0,5}/s$
R_h hydraulic radius in m
k_n wall roughness in m

Veldkamp graph:



Part 1 - Drinking water

The process scheme of the Water Plant Scheveningen (Dunea) is given below.



Processes: softening – dosing of activated carbon - cascades – rapid filters – slow sand filters

1 Describe the function of every treatment step, indicate (quantitatively) which parameters will change per process and give reasons for the order of the processes (why is softening the first step, then followed by PAC, etc).

2 Give a sketch of the hydraulic line of the plant and give a rough estimate for the hydraulic losses in the processes.

3 The O_2 concentration of the drinking water is too low. As an alternative to the cascades, the use of tower aerators is considered. Calculate the efficiency of a co-current tower aerator (use k_2 = 0.4 s^{-1} and t = 4 s) for an RQ of 4 .

4 Give advantages and disadvantages of this alternative as compared to the cascades? Can you think of other alternatives to improve the O_2 concentration?

5 The composition of the raw water is: Ca = 2.0 mmol/L, Mg= 0.25 mmol/L, Na= 0.5 mmol/L, HCO_3= 3.5 mmol/L, CO_2= 0.25 mmol/L.
Which chemical should be used for softening from the point of view of water quality, $Ca(OH)_2$ or NaOH? What other considerations will influence the choice?.

6 Choose from the statements below the one which is the closest to the truth cq the most relevant?

a. The hardness after the pellet reactors is determined primarily by: superficial velocity or bed height or chemical dosing or grain size
b. The energy consumption of the pellet reactors determined primarily by superficial velocity or bed height or chemical dosing or grain size
c. The investment cost of the pellet reactors is determined primarily by: superficial velocity or bed height or chemical dosing or grain size
d. The use of garnet sand as alternative to silica sand for seeding material will lead to higher head loss or lower expansion or higher sand costs or lower chemical costs.

7 For the treatment of backwash water a sedimentation tank is used with dimensions:
H=1 m, L=10m, B=10m. The average flow of the backwash water is 100 m^3/h.
Check whether the design complies to the criteria of surface load (s<1 m/h), turbulence (Re<2000) en stability (Cp>10^{-5})?

8 Four alternatives are being considered:
H = 1 m, L = 50 m, B = 2 m
H = 1 m, L = 10 m, B = 5x2 m (4 intermediate walls)
H = 5x0.2 m, L = 10 m, B = 10 m (4 intermediate floors)
Compare the alternatives and the original design, give advantages and disadvantages. Which design do you prefer?

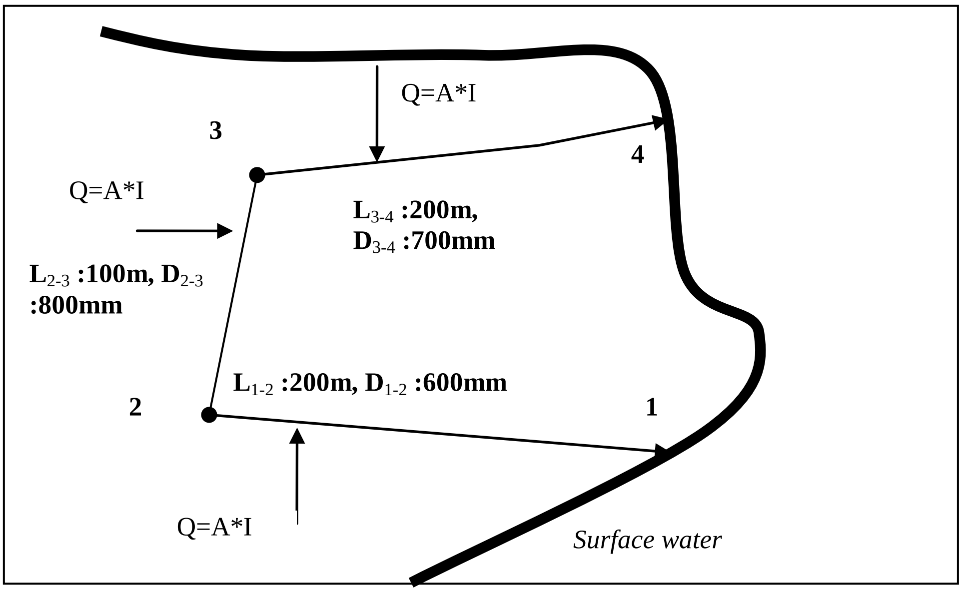
9 The rapid sand filters have the following specifications:
v = 10 m/h
d = 0.6 mm
L = 1.0 m
v backwash = 20 m/h
Calculate the head loss during backwashing.
What will be the effect of increasing the v backwash to 40 m/h on head loss and expansion?

10 The backwashing does not function properly, mud balls are being formed in the filter.
What will be the effect of increasing d to 0.8 mm on the head loss during backwashing, the backwash velocity and the cleaning of the filter?

(*each question 10 points*)

Part 2 - Urban drainage / Sewerage

The lay-out of a combined sewer system is given in the figure below:.



Layout of combined sewer system

The following design conditions apply:

- wall roughness (Nikuradse)	:	k_n = 2 mm
- pipe profile	:	round
- rainfall intensity (I)	:	90 L/(s.ha)
- wastewater production	:	10 L/inhabitant/h
- number of inhabitants	:	300
- connected surface area (A) section 1-2	:	2 ha
- connected surface area (A) section 2-3	:	6 ha
- connected surface area (A) section 3-4	:	4 ha
- weir coefficient	:	1.4 $m^{0.5}$/s

1 What is the total overflow discharge (in m^3/s or m^3/h) to surface water if it rains continuously with the given rainfall intensity and there is no pumping station ? (*10 points*)

2 Calculate the overflow frequency based on the above system characteristics and a pumping station with a capacity of 60 m^3/h for rainfall ("pump-overcapacity" of 60 m^3/h). (*20 points*)

3 Calculate the discharge in each of the sections, assuming that the energy level in node 1 and 4 is the same. (*40 points*)
(P.S. first explain the calculation steps, then proceed to make the calculation)

4 If the discharge in section 3-4 is 0.5 m^3/s, calculate the required width of the overflow weir in node 4, if the surface water level is 0.0 m+NAP and the ground level in node 4 is 0.5 m+NAP. (*30 points*)

Part 3 - Treatment of wastewater

Primary sedimentation

The average wastewater flow to a WWTP is 50,000 m^3/day. The maximum flow rate of the wastewater is 6,000 m^3/h. Following bar screens and grit removal the wastewater is treated in (a) primary sedimentation tank(s).

For primary sedimentation the following constants and formulas can be used:

- hydraulic surface loading rate	: v_0 = 2 – 4 $m^3/(m^2.h)$
- minimum retention time	: $t_{min} \approx 1$ h
- critical scouring velocity	: v_s = 0.30 m/s (sand)
	: v_s = 0.03 m/s (primary sludge)
	: v_s = 0.02 m/s (activated sludge)
- maximum weir loading	: 10 – 15 $m^3/(m.h)$

1 Determine the diameter D and average depth H of (a) circular sedimentation tank(s), considering number of tanks, hydraulic surface loading rate, retention time, scouring, and weir loading. (*20 points*)

Biological treatment

A conventional activated sludge plant, consisting of an aeration tank and secondary settler receives domestic sewage from a Dutch town "X" with an average dry weather flow (adwf) of 10,000 m^3/d and an average BOD of 250 mg/L. The volume of the aeration tank is 4,200 m^3 and the concentration of mixed liquor suspended solids in the aeration tank is 4 g/L. (*5 points (question 2), rest 10 points)*

2 What would be the approximate number of inhabitants in town "X"? Explain your guess.

3 Calculate the organic sludge loading rate in the activated sludge plant of town "X".

4 Would the town "X" sewage treatment plant be a nitrifying activated sludge plant? Explain.

5 Explain why in an activated sludge plant the organic loading rate and not the nitrogen loading rate determines whether nitrification will take place.

6 Define sludge age using a formula and explain, in words, how sludge age is related to organic loading rate.

7 The actual oxygenation capacity of an aeration system is given by: $OC_{act} = \alpha\, OC \frac{C_S - C}{C_S}$

Explain the meaning of OC_{act} and the symbols in this equation. Discuss what would be the optimal dissolved oxygen (DO) concentration in terms of oxygenation capacity. Do you expect a low or high nitrification rate at this DO?

8 For the design of secondary settling tanks the following equation is used:

$SVLR = V_o X_{AT} \frac{SVI}{1000}$, where V_o = surface loading rate (m/h), X_{AT} = suspended solids concentration in aeration tank (gMLSS/L) and SVI= sludge volume index (mL/g). The design value for the Sludge Volume Loading Rate (SVLR) is 0.3 m/h. A conventional activated sludge system consists of an aeration tank and a circular secondary settling tank. The influent flow is 5,000 m^3/d, the sludge recirculation factor is 0.5 and the suspended solids concentration in the aeration tank is 4 gMLSS/L. Calculate the design diameter of the settling tank when the SVI is expected to be 100 mL/g.

Sludge treatment

A flow of 750 m^3/day of waste sludge (1% SS of which is 70% organic matter) is thickened to a SS-concentration of 50 gSS/L by gravity. The thickened sludge is pumped to a digester. In the digester 50% of the organic matter is degraded to biogas. The digested sludge is dewatered to 25% SS (no chemicals are needed for dewatering).

9 How many trucks with a capacity of 30 tons are needed per week to transport the dewatered sludge to the central incineration plant? (*15 points*)

ANSWERS

Part 1 - Drinking water

1 Softening lowering of hardness by precipitation of $CaCO_3$, first step in order to coprecipitate Fe, prevent additional filter step and additional pump phase.
PAC for removal of pesticides and odour, taste and color compounds, second step so the PAC can be removed in the sand filters.
Cascades third step to increase oxygen and oxidize Fe, Mn and NH_4. Robust system in view of PAC and fouling of Fe.
Sand filters to remove Fe, Mn, NH_4 and carry-over from pellet reactors.
SSF for final polishing, removal of particles and micro-organisms.

2 Hydraulic line includes just one pump step from low point in the dunes (approx NAP, 5 m below GL) to high point in the pellet reactors (10 m above GL).
Head loss in piping 2 m, head loss in pellet reactor 4 m, head loss in cascades 2 m, head loss in sand filters 2 m, head loss in SSF 2 m, clear water tank approx GL (min 5 m below, max 0).

3

$$K_3 = \frac{1 - e^{\left(-k_2 \cdot t\left(1+\frac{k_d}{RQ}\right)\right)}}{1 + \frac{k_d}{RQ}}$$

Substitute: $k_2t = 1.6$, $k_d/RQ = 0.01$, so K will become 0.8

4 Advantage that it has a high efficiency and will fit in the hydraulic line, disadvantage is clogging by the PAC.
Alternative could be to use small cascade or bubble aeration after the sand filter or even after the SSF.

5 Ca $(OH)_2$ is possible, required dosing is 0.75 mmol/L for softening and approx. 0.25/2 mmol/l for CO2 removal, drinking water quality will be Ca = 1.25 mmol/L, Mg= 0,25 mmol/L, HCO_3= 3.5+0.25-2*0.75= 2.25 mmol/L, is fine.
NaOH will lead to higher HCO_3 and higher Na, also lower pH.
Other considerations are costs and difficulties in handling of $Ca(OH)_2$.

6 The hardness after the pellet reactors is determined primarily by the chemical dosing
The energy consumption is determined primarily by the bed height
The investment cost is determined primarily by the superficial velocity
The use of garnet sand leads to lower chemical costs

7 s = 1 m /h, R = 10x1/(10+2) = 0.88. v = 10/3600 = 0.3x 10^-2 m/s.
Re = 0.3x10^-2x0.88/10-6 = 2700, so >2000.
Cp = (0.3x10^-2)^2/10x0.88 = 10^-6, so <10^-5

8 First alternative improves stability somewhat, second alternative improves stability also, third alternative is superior regarding surface load and turbulence. Disadvantage is cost and sludge handling.

9 Head loss is 1 m H_2O, increasing v backwash has no effect in head loss, but increases the expansion

10 Increasing d to 0.8 mm is good, head loss will remain the same, but v backwash will increase and cleaning efficiency will be much better.

Part 2 - Urban drainage / Sewerage

1 Stormwater: section 1-2 : 180 l/s, Section 2-3 540 l/s, Section 3-4 360 l/s, Total 1080 l/s
Wastewater: 1 l/s
Total flow: 1081 l/s or 3892 m^3/h

2 Section 1-2: D=600, L=200, storage = 56.6 m^3
Section 2-3: D=800, L=100, storage = 50.3 m^3
Section 3-4: D=700, L=200, storage = 77.0 m^3
Total storage = 183.8 m^3 or 1.52 mm (12 ha)
Pump over capacity of 60 m^3/h equals 0.5 mm/h.
Veldkamp graph: Overflow frequency: 80/year

3 Calculation steps:
- estimate flow per conduit based on connected area and rainfall intensity
- calculate energy losses and check if sum of energy losses over the loop is 0
- if sum of energy losses is not 0: adjust according to Cross method (Newton Raphson approximation)

First estimate flows in conduit 1-2, 2-3 and 3-4:

	Q	L/C2RA2	ΣHdyn	a*Q	dQ	**Q**	L/C2RA2	ΣHdyn	a*Q	dQ
1-2						**-0,18**	5,898	-0,191	1,062	
2-3						**0,54**	0,644	0,188	0,348	
3-4						**0,9**	2,609	2,114	2,348	
						-1,08		**2,110**	3,758	-0,281
1-3	**-0,461**	5,898	1,252	2,718		**-0,438**	5,898	1,130	2,581	
3-2	**0,259**	0,644	-0,043	0,167		**0,282**	0,859	-0,068	0,243	
1-2	**0,619**	2,609	-1,000	1,616		**0,642**	3,044	-1,256	1,956	
			0,208	4,500	0,023			-0,195	4,779	-0,020
1-3	**-0,458**	5,898	1,237	2,702		**-0,438**	5,898	-1,129	2,581	
3-2	**0,262**	0,644	-0,044	0,169		**0,282**	0,644	0,051	0,182	
1-2	**0,622**	2,609	-1,009	1,623		**0,642**	2,609	1,077	1,676	
			0,184	4,493	0,020			-0,001	4,439	0,000
	-0,438	5,898	-1,130	2,582						
	0,282	0,644	0,051	0,182						
	0,642	2,609	1,076	1,676						
			-0,002	4,439	0,000					

$Q_{1\text{-}3}$ = -0.44 m^3/s, $Q_{3\text{-}2}$ = 0.28 m^3/s, $Q_{1\text{-}2}$ = 0.62 m^3/s,

4 Overflow = 0.5 m^3/s, CC = 1.4, H_{max} = 0.4 m
assume: overflow weir 10 cm above surface water level
$H^{3/2}$ = 0,25, b = 1.41 m

Part 3 - Treatment of wastewater

Primary sedimentation

1 The minimum number of tanks is two (2).

at Q_{max}: v_0 = 4 m³/(m².h) ➔ A_{tot} = 6,000/4 = 1,500 m²
➔ A_{tank} = 1,500/2 = 750 m² ➔ D_{tank} = $(4.750/\pi)^{½}$ = 31 m

at Q_{max}: $v_s = \frac{Q_{max}}{\pi \cdot x \cdot H_{tank}} \cdot \frac{1}{3,600}$ (m / s) (x = distance from middle of tank, take x = 2m)

$$H_{tank} = \frac{3,000}{\pi \cdot 2 \cdot 0.03} \cdot \frac{1}{3,600} = 4.4 \text{ m}$$

volume: V_{tank} = ¼•π•31²•4.4 = 3321 m³

at Q_{max}: t = 3321/3000 = 1.1 h

at Q_{AVG}: $t = \frac{3321}{\frac{1}{2} \cdot 50,000 \cdot \frac{1}{24}} = 3.2 \text{ h}$

at Q_{AVG}: $v_0 = \frac{\frac{1}{2} \cdot 50,000 \cdot \frac{1}{24}}{\frac{1}{4} \cdot \pi \cdot 31^2} = 1.4$ m³ /(m².h)

at Q_{AVG}: $v_s = \frac{\frac{1}{2} \cdot 50,000 \cdot \frac{1}{24}}{2 \cdot \pi \cdot 4.4} \cdot \frac{1}{3,600} = 0.01 \text{ m / s}$

Weir loading:

at Q_{max}: $\frac{3,000}{\pi \cdot 31} = 30.8$ m³ /(m.h)
➔ depending on the duration of overloading consider a double weir

at Q_{AVG}: $v_0 = \frac{\frac{1}{2} \cdot 50,000 \cdot \frac{1}{24}}{\pi \cdot 31} = 10.7$ m³ /(m.h)

Biological treatment

2 Average water consumption per inhabitant is about 125 l/d, and this is roughly the amount of sewage produced. An amount of 10,000 m^3/d would then be produced by 10,000/0.125=80,000 people. Not included is the contribution of water from industries (minor portion in case of domestic sewage) and precipitation (peak loads).

3 10,000 [m^3/d]×0.250 [kgBOD/m^3]/(4200 m^3×4 kgMLSS/m^3])=0.15 kgBOD/kgMLSS.d

4 Sludge loading rate B_X is 0.15 kgBOD/kgMLSS.d, and this is low enough (i.e. <0.20) to allow growth and retention of nitrifying biomass. Hence it is likely a nitrifying plant.

5 At high organic loading rate sludge production is higher, because of high growth of heterotrophic micro-organisms and high input of influent solids. In order to maintain MLSS at a reasonable level (to avoid oxygenation limitations) the sludge wastage rate must be higher. Nitrifiers are slower growers than heterotrophic micro-organisms and will therefore be washed out of the system. At low organic loading rate heterotrophic growth and hence wastage rate is less, so that nitrifiers will be retained.

6 Sludge age is the average residence time of suspended solids in the system: $\theta_X=V_{AT}\times X_{AT}/P_X$ or $\theta_X=V_{AT}\times X_{AT}/(Q_S\times X_S)$. Sludge production rate or sludge wastage rate is proportional to organic loading rate and hence sludge age is inversely proportional to organic loading rate.

7 OC_{act} is the actual amount of oxygen in kgO_2 that is introduced per unit of volume and unit of time by a certain aeration system into the activated sludge tank. OC is the amount of oxygen in kgO_2 that is introduced per unit of volume and unit of time by a certain aeration system into water having an oxygen concentration of zero. α is a correction factor for oxygen mass transfer in wastewater (α <1) instead of clean water (α =1). C_S is the saturation concentration of dissolved oxygen and C is the actual dissolved oxygen concentration. The optimal DO is zero, because then the OC_{act} is maximum. Minimal DO for nitrification is about 2 mg O_2/l, so nitrification rate will be nil at DO=0.

8 Vo follows from $0.3=Vo\times4\times100/1000 \rightarrow Vo=0.75$ m/h or 18 m/d. Recirculation factor $Q_R/Q_I=0.5$. Return sludge flow $Q_R=0.5\times Q_I=0.5\times5000=2500$ m^3/d. Total flow to settling tank Q is $Q_I+Q_R=5000+2500=7500$ m^3/d. Surface area A of settling tank follows from: $Vo=Q/A \rightarrow A=Q/Vo=12500/18=417$ m^2. This corresponds to a diameter of $2\times\sqrt{(694/\pi)}\approx23$m.

Sludge treatment

9 750 m^3 sludge/day (1% SS, 70% organic)
- ➯ 7,500 kgSS/day
- ➯ 742,500 kg water/day (L water/day)

after thickening (50 gSS/l):
- ➯ 7,500 kSS/day
- ➯ 7,500/0.05 = 150.000 kg sludge/day (L sludge/day)
- ➯ 150,000 – 7,500 = 142,500 kg water/day (L water/day)

after digestion (50% of organics):
- ➯ 0.3x7,500 + 0.5x0.7x7,500 = 4.875 kg SS/day
- ➯ still 142,500 kg water/day
- ➯ 147,375 kg sludge/day (L sludge/day) (3.3% SS)

after dewatering to 25% SS:
- ➯ 4,875 kgSS/day
- ➯ 4.875/0.25 = 19,500 kg sludge/day

➔ per week 19.5 x 7 = 136.5 tons ➔ 136.5 / 30 = 4.6 trucks/week