

TU Delft
Faculty of Civil Engineering and Geosciences
Section Sanitary Engineering

Exam CT3420 - Sanitary Engineering

Date : 18 August 2009
Time : 9.00 - 12.00

The exam consists of 3 parts: drinking water, sewerage and wastewater treatment. Each part counts for 33% in the final score.

The objective for all questions is to show that you understand the course material. Always motivate your answer and ask yourselves whether the answer is complete.

An A4 sheet with your own notes is NOT permitted, a formula sheet with relevant formulas is added.

If there are ambiguities on the issue, report it to avoid confusion.

For each section use a separate answer sheet. Write your name and student number on every sheet.

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·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 :

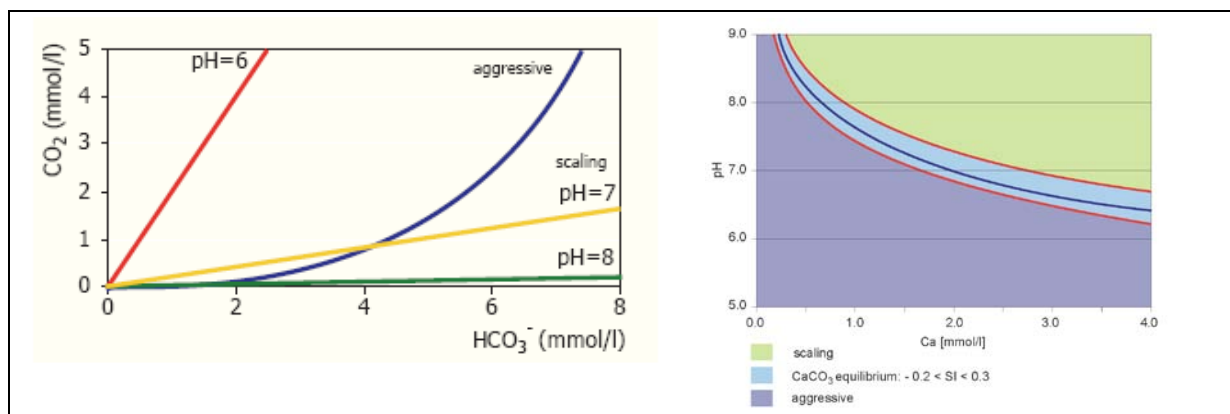
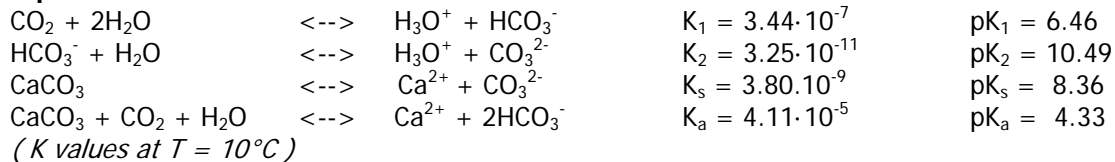


Figure 1. Tillmans-curves (on the left the relation $\text{CO}_2\text{-HCO}_3^-$, on the right the relation Ca-pH)

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:	$c_g = p_a / (RT) \quad R = 8,3143 \text{ J mol}^{-1} \text{ K}^{-1}$
Henry's law:	$c_s = K_d \cdot c_g \quad (\text{mol/m}^3)$

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:	$\Delta H_v = \sum \xi_i \cdot \left(\frac{v^2}{2 \cdot g} \right)$
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)

<p>Relevant formulas aeration / degassing:</p> $K_1 = 1 - e^{(-k_2 \cdot t)}$ $K_2 = \frac{1}{1 + \frac{1}{k_2 \cdot t}}$ $K_3 = \frac{1 - e^{(-k_2 \cdot t \cdot (1 + \frac{k_d}{RQ}))}}{1 + \frac{k_d}{RQ}}$ $K_4 = \frac{1 - e^{(-k_2 \cdot t \cdot (1 - \frac{k_d}{RQ}))}}{1 - \frac{k_d}{RQ} \cdot e^{(-k_2 \cdot t \cdot (1 - \frac{k_d}{RQ}))}}$ $K_5 = \frac{1}{1 + \frac{1}{k_2 \cdot t} + \frac{k_d}{RQ}}$	<p>Relevant formulas filtration:</p> $l_0 = \frac{H_0}{L} = 180 \cdot \frac{v}{g} \cdot \frac{(1 - p_0)^2}{p_0^3} \cdot \frac{v}{d_0^2}$ $H = 130 \cdot \frac{v^{0.8}}{g} \cdot \frac{(1 - p_e)^{1.8}}{p_e^3} \cdot \frac{v^{1.2}}{d^{1.8}} \cdot L_e$ <p>Relevant formulas sedimentation:</p> $v_s = \frac{1}{18} \cdot \frac{g}{v} \cdot \frac{\rho_s - \rho_w}{\rho_w} \cdot d^2$ $Re = \frac{v_o \cdot R}{v}$ $c_p = \frac{v_o^2}{g \cdot R}$
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Formule sheet CT3420 - Sewerage

Overflow formula:

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

Where:

- Q flow in m³/s
m overflow coefficient in m^{0.5}/s
h thickness using overflow radius in m

Local losses:

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

Where:

- ΔH loss of energy height in m
 ξ Loss coefficient (dimensionless)
Q flow in m³/s
A surface of the wet section in m²
g gravitational acceleration in m/s²

Friction loss in a pipe:

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

Where:

- ΔH loss of energy height m
C Chezy coefficient in m^{0.5}/s
Q flow in m³/s
L length of the pipe
 R_h hydraulic radius in m
A wet surface diameter in m²

The hydraulic radius R_h is defined as:

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

where:

- A wet surface diameter in m²
P wet perimeter in m

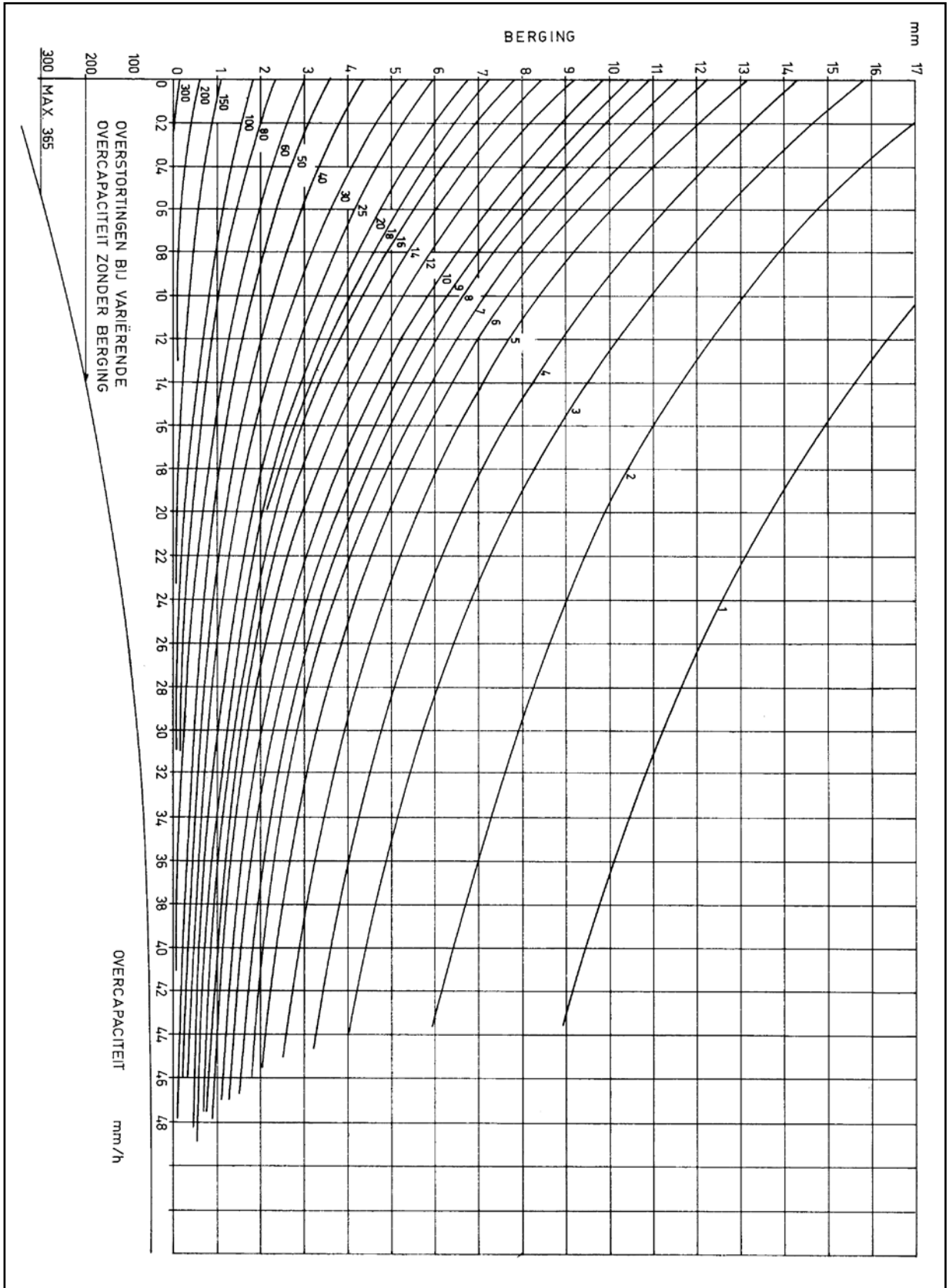
The Chezy coefficient is defined as:

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

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

1. Water quality and water chemistry

Duinwaterbedrijf Zuid-Holland (DZH) abstracts surface water from a dammed branch of the river Meuse (Maas) called the "Andelse Maas basin". At the intake point the water passes microsieves. Thereafter the water is transported to Bergambacht where the water is purified by rapid sand filtration. Bergambacht is located at a distance of 30 km from the abstraction point at the "Andelse Maas basin". After the treatment of the water at Bergambacht, the water is transported to the dunes near Scheveningen. In these dunes the water is infiltrated, and after approximately 3 months the water is extracted from the dunes and collected in an uncovered basin, where powdered activate carbon is added. Then the water is softened, aerated and then filtered within rapid and slow sand filters. Hereafter the water is distributed in the city of The Hague and surrounding area. Figure 1 shows the process diagram schematically.

Figure 1 – Schematic view of water treatment processes for drinking water production from the Andelse Maas basin.

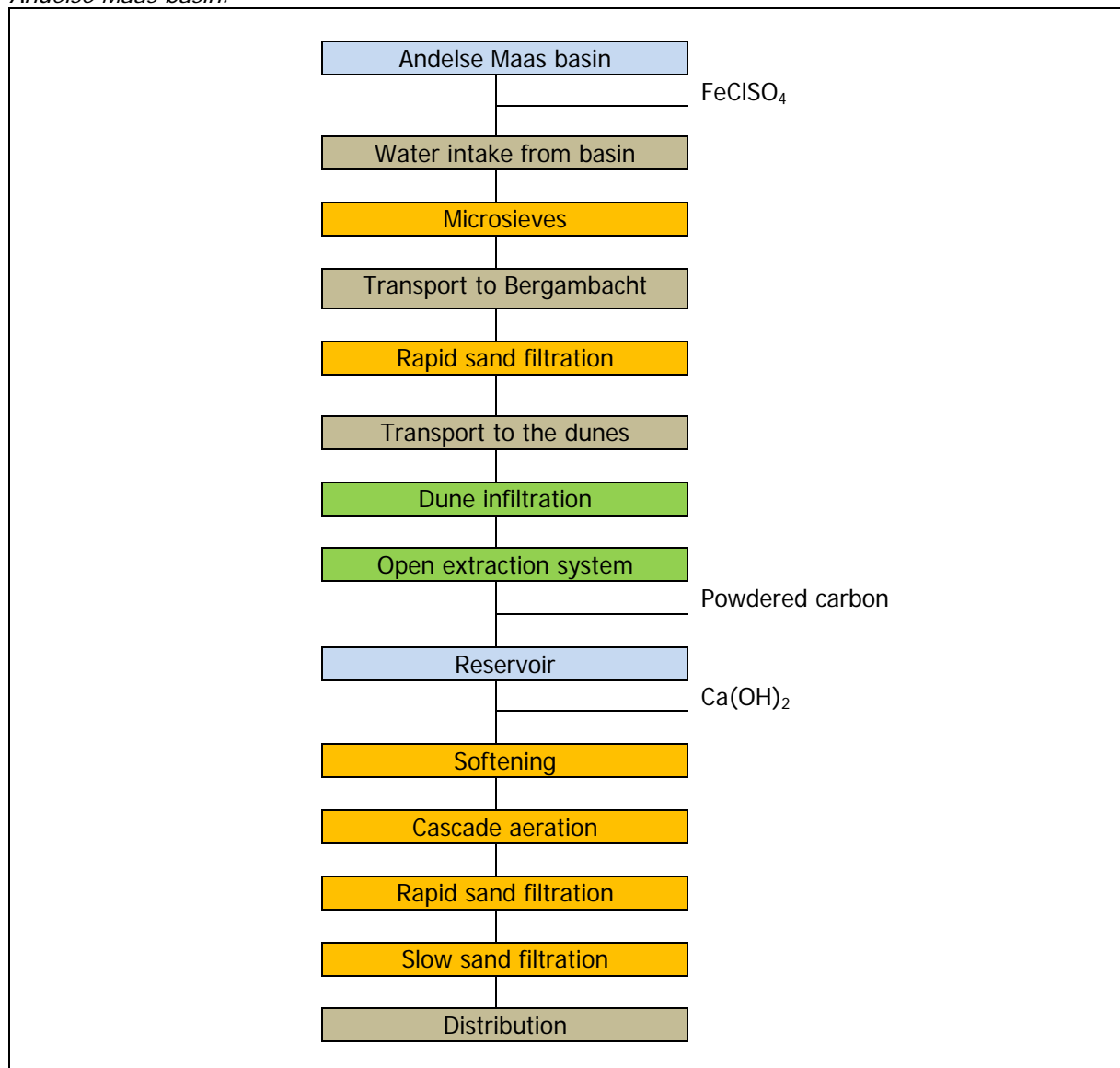


Table 1 shows the average quality of the water at various places in the purification process.

Table 1 - Quality of dune water

Parameter	Unit	Maas	Bergambacht after rapid filtration	Open extraction	Drinking water
Turbidity	FTU	10	1	2	0
O ₂	mg/l	10	10	1	7
Fe ²⁺	mg/l	0	0	2	0
NH ₄ ⁺	mg/l	0	0	1	0
NO ₃ ⁻	mg/l	3	3	0	3
pH	-	7.9	7.6	7.3	8.6
CO ₂	mg/l	1	2	4	0
HCO ₃ ⁻	mg/l	190	188	200	150
Ca ²⁺	mg/l	67	67	72	52
E-coli	n/l	10 ⁶	10 ⁴	10 ²	0
BM	µg/l	0.5	0.5	0.3	0

- 1.1 Describe in words the processes responsible for the decrease in the turbidity from 10 FTU to 1 FTU, rising to 2 FTU and falling back to 0 FTU.
- 1.2 Describe in words the processes responsible for the decrease in O₂-concentration from 10 mg/l to 1 mg/l and the increase to 7 mg/l.
- 1.3 Describe in words the processes responsible for the increase in Fe-concentration to 2 mg/l and decrease again to 0 mg/l
- 1.4 Calculate how much O₂ is used for the oxidation of NH₄⁺ from the water in open dune extraction.
- 1.5 Describe in words the processes responsible for the decrease in NO₃⁻-concentration from 3 mg/l to 0 mg/l and the increase to 3 mg/l.
- 1.6 Describe in words the processes responsible for the decrease in pH from 7.9 to 7.6 to 7.3 and and the increase to 8.6.
- 1.7 Describe the theory of the similarity of the decrease in pH and the increase in CO₂-concentration between "Maas" and "Bergambacht".
- 1.8 Calculate whether the decrease in Ca²⁺-concentration and the HCO₃⁻-concentration between the open water extraction and the drinking water are correct.
- 1.9 Describe in words the processes responsible for the course of the concentration of E-coli.
- 1.10 Describe in words the processes responsible for the variation of the concentration of pesticides.

2. Drinking water treatment

These questions are also related to the water treatment at Duinwaterbedrijf Zuid-Holland (DZH)

- 2.1 What are the functions of damming the "Maas basin"?
- 2.2 What is the function of FeClSO_4 -dosage?
- 2.3 What is the function of microsieves?
- 2.4 Why is the rapid filtration located at Bergambacht and not near the intake?
- 2.5 What is the function of the dunes?
- 2.6 What is the disadvantage of dosing powdered activated carbon **after** the dune infiltration?
- 2.7 Recently the water company switched to closed extraction. Give the implications of this change for the quality, run time, and the filtration velocity of the rapid sand filtration and the slow sand filtration.

Part 2 - Sewerage

Given a mixed sewerage system as shown in Figure 1.

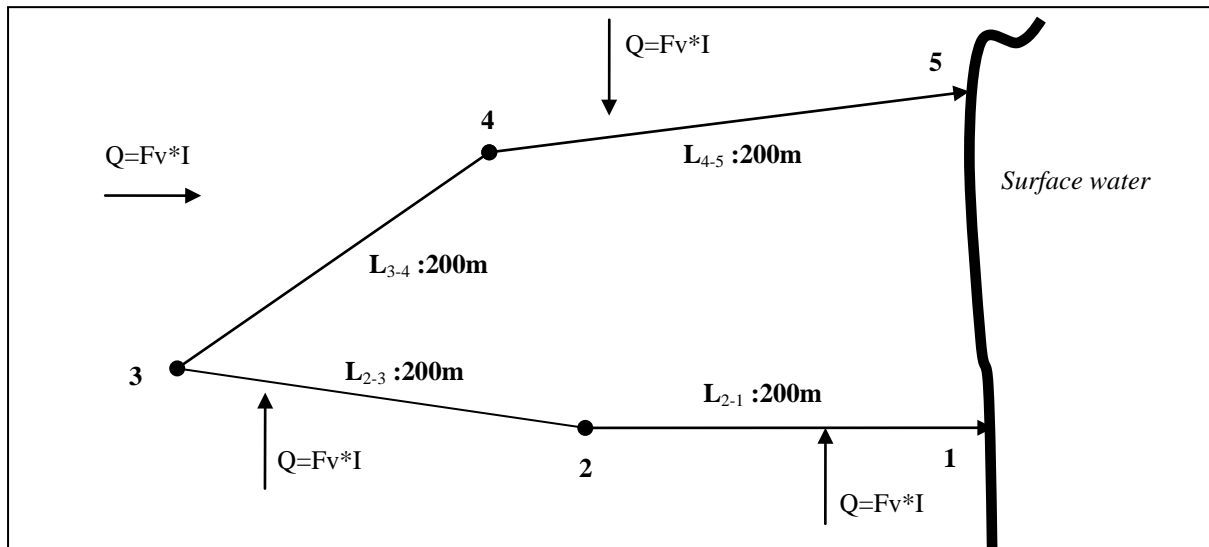


Figure 1

Equivalent sand roughness acc. Nikuradse	:	$k_n = 2 \text{ mm}$
Pipe profile	:	spherical
Rainfall intensity (I)	:	60 l/(s.ha)
Population	:	300
Draining surface (Fv) 1-2 strict	:	6 ha
Draining surface (Fv) 2-3 strict	:	2 ha
Draining surface (Fv) 3-4 strict	:	4 ha
Draining surface (Fv) 4-5 strict	:	4 ha
Overflow coefficient	:	$1.4 \text{ m}^{0.5}/\text{s}$

Wanted:

- 1 a. What is the total stormwater flow to the surface water when it rains with given rainfall intensity?
Assumption: storage capacity of the system is negligible.
- 1 b. How large should be the storage in the system to achieve a frequency of overflow of 7x/year ?
2. Assume that storage is achieved according to the answer to question 1b.
The real overflow rate for the past 5 years is higher than expected from the storage capacity. Give 2 possible causes of the higher overflow rate than on the basis of a theoretical approach (Dots graph or Veldkamp graph) is expected.
3. Calculate the distribution of the flow over the pipe branches given that:
 - pipe 1-2 has a diameter of 600 mm
 - pipe 2-3 has a diameter of 700 mm
 - pipe 3-4 has a diameter of 700 mm
 - pipe 4-5 has a diameter of 600 mm
 (P.S. first explain how you will handle this question, then you can start calculating)
4. Assuming that the flow in pipe 1-2 is $0.45 \text{ m}^3/\text{s}$, determine the necessary width of the overflow weir at point 1 if given that the level of the receiving surface water is $0.0 \text{ m} + \text{NAP}$ and the ground level at the point 1 is $0.5 \text{ m} + \text{NAP}$.
(diameters of the pipes are identical to those given in Question 3).

Part 3 - Treatment of wastewater

1. Aerobic treatment

The wastewater treatment plant (wwtp) Houtrust in The Hague is a conventional treatment plant from the 60's. During the past 25 years the plant is modified several times to meet the increasingly stringent effluent requirements.

Q dwa-average	= 220,000 m ³ /d
Q rwa-max	= 39,600 m ³ /h
Influent BOB	= 350 mg/l
COD/BOD ratio	= 2:1

Initially the treatment process was only a pre-sedimentation tank followed by a discharge pipe to the sea. The efficiency of this primary sedimentation is 40%. Subsequently, a biological treatment has been added. From the 80's aeration was done by adding pure oxygen. The system consisted of pre-sedimentation – aeration tanks - sedimentation - sea emission. The aeration tanks have a total volume of 20,000 m³.

The questions focus on an average dry-weather situation unless otherwise stated.

- 1.1 The average sludge concentration in the aeration tanks is about 3 – 3.5 kg MLSS/ m³. Show by calculation whether this is a a) high rate, b) moderate rate or c) low rate wastewater treatment system.
- 1.2 Calculate the applied sludge return flow (Q_r) if a sludge concentration X_r of 9 g/l is measured in the return flow
- 1.3 How nitrogen (N) is removed in this system?
- 1.4 Calculate the maximum N removal (kg N/d) in the aeration tank assuming a sludge yield of 0.5 kg VSS / kg COD removed and a specific N requirement for growth of 0.12 kg N / kg VSS.
- 1.5 Give an estimate of the maximum achievable N removal efficiency in the aeration tank (%) relative to the total N load of the wwtp Houtrust.
- 1.6 In 1990 the wwtp has been extended with a sludge digestion plant. Indicate which processes take place in the sludge digestion and what are the 3 main reasons to apply sludge digestion.
- 1.7 In general, the sludge reject water is returned to the main wwtp after sludge digestion. Explain the impact of this reject water on nitrogen removal in the treatment plant?
- 1.8 Which adjustment(s) of the system do you propose to optimize the N removal and on which location of the treatment you would apply this ? Which modifications should be provided at the plant to achieve this?

2. Anaerobic treatment

South of Cairo a waste water treatment plant (wwtp) is newly built for a flow of $7.200 \text{ m}^3/\text{dag}$. The COD concentration is $1,000 \text{ mg/l}$.

Because of energy considerations, the city has chosen for an anaerobic treatment plant, type UASB as alternative.

The draft principles are:

- average hydraulic residence time (θ) = 10 hours
- average sludge load = $0.05 \text{ kg COD/kg MLSS.day}$

- 2.1 Calculate the needed volume of the UASB, based on the applied residence time and based on an average sludge concentration of 40 kg MLSS/m^3 . Adjust the height of the reactor at 5 m and keep the volumetric surface load below of 0.7 m/h .

Part 1 - Drinking water

1. Water quality and water chemistry

- 1.1 Turbidity decreases from 10 FTE to 1 FTE by sedimentation in "Andelse Maas basin" (promoted by dosage of FeClSO_4 en filtration in rapid sand filters at "Bergambacht". The turbidity increases then from 1 FTE to 2 FTE by dissolving of Fe in the dunes subsurface and afterwards the flocculation of Fe in the collecting reservoir. The turbidity decrease of 2 FTE to 0 FTE by rapid sand filtration and slow sand filters.
- 1.2 The oxygen concentration drops from 10 mg / l to 0 mg / l by the decomposition of organic matter in the soil, then it rises to 1 mg / l by a little aeration in the collecting reservoir. Then it rises back again to 10 mg/l because of the aeration cascade, but it decreased again by oxidation of NH_4^+ .
- 1.3 Dissolution of pyrite in the soil, then oxidation of Fe^{2+} by O_2 in the cascades and flocculation to $\text{Fe}(\text{OH})_3$ in the rapid sand filters.
- 1.4 $2 \cdot 32 / 17 = 3.5 \text{ mg/l}$
- 1.5 The NO_3^- - concentration drops because of biological denitrification in the soil and increases because of the oxidation van NH_4^+ in the rapid sand filters.
- 1.6 Firstly the dose of FeClSO_4 , then the biological degradation in the soil and finally the dosage of $\text{Ca}(\text{OH})_2$ and the softening.
- 1.7 pH reduction of 0.3 corresponds to increasing of CO_2 by a factor 2 according to the acid equilibrium.
- 1.8 Drop of Ca = 20 mg/l , ie 0.5 mmol/l, so HCO_3^- should be decrease $2 \cdot 0.5 \cdot 61 = 61 \text{ mg/l}$, is not correct.
- 1.9 Firstly the drop by sedimentation and filtration in Bergambacht, then drop by infiltration, but again increasing in collecting reservoir, then drop by slow sand filtration.
- 1.10 Decrease by adsorption in the soil and subsequently by adsorption on powdered activated carbon.

2. Drinking water treatment

- 2.1 Especially self-purification (especially sedimentation), also known as leveling and analysis reservoir, not because of a storage function.
- 2.2 P removal to prevent algae growth.
- 2.3 Removal of mussel larvae and growth control in transport pipelines.
- 2.4 Historically development, is not really good, though you have now the possibility of an additional intake from the Lek.
- 2.5 Storage, leveling, microbiological reliability, quality improvement, robustness
- 2.6 Loading of the dunes with pesticides.
- 2.7 Because of the better quality it is possible to achieve longer running times, higher filtration rates and better quality at the rapid sand filters, even the slow sand filtration is now completely unnecessary.

Part 2 - Sewerage

1a

Rainfall intensity	60	l/s/ha
Disposal area	16	Ha
Drain flow	960	l/s

1b

7x/year overflow frequency

Veldkamp graph: 11.5 mm storage (assume that poc is 0)

2

Causes of higher than theoretical (dots / Veldkamp) approach:

- Sedimentation in the system - part of storage available
- Subsidence of pipes which water will remain permanently – part of storage is unavailable.
- Rainfall in the past 5 years is higher than for assumptions which Veldkamp graph is based on: other periods and location than rainfall data used Veldkamp graph
- Paved surface is increased in recent years which make the resulting storage smaller (in mm) than under the original design.
- Sewerage system is not constructed according to design: wrong pipe diameters etc.

3

The mesh is symmetrical, the inflow almost symmetrical: symmetry assumption is a good approach. Cut the mesh in half at node 3 and calculate the discharge flows.

Or: Method Cross

Starting point: fully filled pipes, take a positive flow direction and do a first assumption for the flow rates in pipes, based on inflow and mass balance (total of incoming flows per node = total of outgoing flows per node).

The sum of the energy losses over the mesh must be equal to 0:

Calculate the energy losses on the pipelines for the assumed flow rates and calculate the sum of energy losses. As sum $\Delta H \neq 0$: adjust the flows with ΔQ and recalculate

	D	L	Fv	Q	A	R	C	L/C2RA2	Hdyn	a*Q	dQ
1-2	600	200	6	0.48	0.283	0.150	53.176	5.898	1.359	2.831	
2-3	700	200	2	0.12	0.385	0.175	54.381	2.609	0.038	0.313	
3-4	700	200	4	-0.24	0.385	0.175	54.381	2.609	-0.150	0.626	
4-5	600	200	4	-0.48	0.283	0.150	53.176	5.898	-1.359	2.831	
Sum:				0.96	480.000				-0.113	6.602	-0.009

	L/C2RA										
	Q	2	Hdyn	a*Q	dQ	Q	L/C2RA2	Hdyn	a*Q	dQ	
1-2	0.471	5.898	-1.311	2.781		0.488	5.898	-1.403	2.877		
2-3	0.111	2.609	-0.032	0.291		0.128	3.044	-0.050	0.389		
3-4	-0.240	2.609	0.150	0.626		-0.240	3.044	0.175	0.731		
4-5	-0.489	5.898	1.408	2.881		-0.472	5.898	1.315	2.785		
			0.215	6.579	0.016			0.038	6.782	0.003	

	L/C2RA										
	Q	2	Hdyn	a*Q	dQ	Q	L/C2RA2	Hdyn	a*Q	dQ	
1-2	0.491	5.898	-1.419	2.893		0.490	5.898	1.413	2.887		
2-3	0.131	2.609	-0.044	0.341		0.130	2.609	0.044	0.338		
3-4	-0.240	2.609	0.150	0.626		-0.240	2.609	-0.150	0.626		
4-5	-0.469	5.898	1.300	2.769		-0.470	5.898	-1.306	2.775		
			-0.014	6.629	-0.001			0.001	6.626	0.000	

4

Overflow flow: 0.45 m³/s

Maximum height overflow radius: distance between ground level and weir (assume the dam height 0.1 m above water level: 0.1m +NAP): 0.5+NAP-0.1+NAP=0.4m

$Q = mBH^{3/2}$

m=1.4, H=0.4, Q=0.45: B=1.27m

Part 3 - Treatment of wastewater

1. Aerobic treatment

- 1.1 Given: $Q_{dwa} = 220,000$, $BOD = 350$, $V=20,000$, $X = 3.25$.
Sludge load (L_x) = $(220,000 * 0.35) / (20,000 * 3.25) = 1.2$ kg BOD/kg MLSS.d
This is seen as a highly loaded system.
- 1.2 $(Q + Q_r) X = Q_r X_r$ $\rightarrow (220,000 + Q_r) * 3.25 = Q_r * 9$
 $Q_r = Q.X / (X_r - X)$ $\rightarrow Q_r = 220,000 * 3.25 / (9 - 3.25) = 124,348$ m³/d
- 1.3 N-removal through:
a) SS settling with organically bound N and primary sludge disposal
b) growth of secondary sludge
- 1.4 COD load : $2 * 0.35 * 220,000 = 154,000$ kg COD/d
After primary sedimentation: $0.6 * 154,000 = 92,400$ kg COD
Sludge growth: $0.5 * 92,400 = 46,200$ kg VSS
N removal: $0.12 * 46,200 = 5,544$ kg N/d
- 1.5 N-total in sewage in NL: $50-60$ mg N/l
Total N load: $11,000 - 13,200$ kg N/d
Maximum N removal eff.: $42-50\%$.
- 1.6 Sludge digestion: Anaerobic degradation of primary and secondary sludge with biogas as an end product.
Reasons for application:
a) anaerobic stabilization of primary and secondary sludge
b) production of biogas as energy for aeration
c) improving dewaterability to reduce transport costs for excess sewage sludge.
- 1.7 During fermentation the organically sludge-bound N is released into the solution. Return of the sludge reject water to the treatment plant increases the N load to the aeration tanks. The result is:
a) additional energy requirement for aeration to oxidize all N
b) reduction of N removal efficiency.
- 1.8 Construction of an anoxic zone / denitrification zone in front of the aeratietank. A so-called pre-denitrification step to where activated sludge is recycled from the aeratietank. The treatment should be complemented with an anoxic tank and sludge return pipelines to the anoxic tank and effluent return line from the aeration tank to the anoxic tank.

2. Anaerobic treatment

- 2.1 $V = Q * HRT$
 $V = 7,200 * 10 / 24 = 3,000$ m³
 $V = 1$ (kg/m³) * $7,200$ (m³/d) / $(0.05 * 40) = 3,600$
- $V_{min.} = 3,600$
- $V = Q * HRT$
 $A * h = Q * HRT \rightarrow V_{upw.} = Q / A = h / HRT = 5 / 10 = 0.5$ m/h....
0.7 is not reached!