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Environmental Engineering Design and Project Management

Detailed design proposal for a wastewater treatment plant in Newtown Aycliffe.

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Executive summary

CASSIE Environmental Consulting was hired to design a wastewater treatment plant for the municipality of Newtown Aycliffe in England. A conceptual proposal was submitted prior to this report to receive feedback from the involved stakeholders. This report represents the second and final step in the consultancy part of the project.
The municipality has a current population of 26,000 people with a growth of around 1% per year. The wastewater treatment plant will receive only domestic discharges estimated at around 3200 m^3/day. In the next two decades daily discharge are expected to reduce due to DEFRA target of 130L/p.e. day discharge.

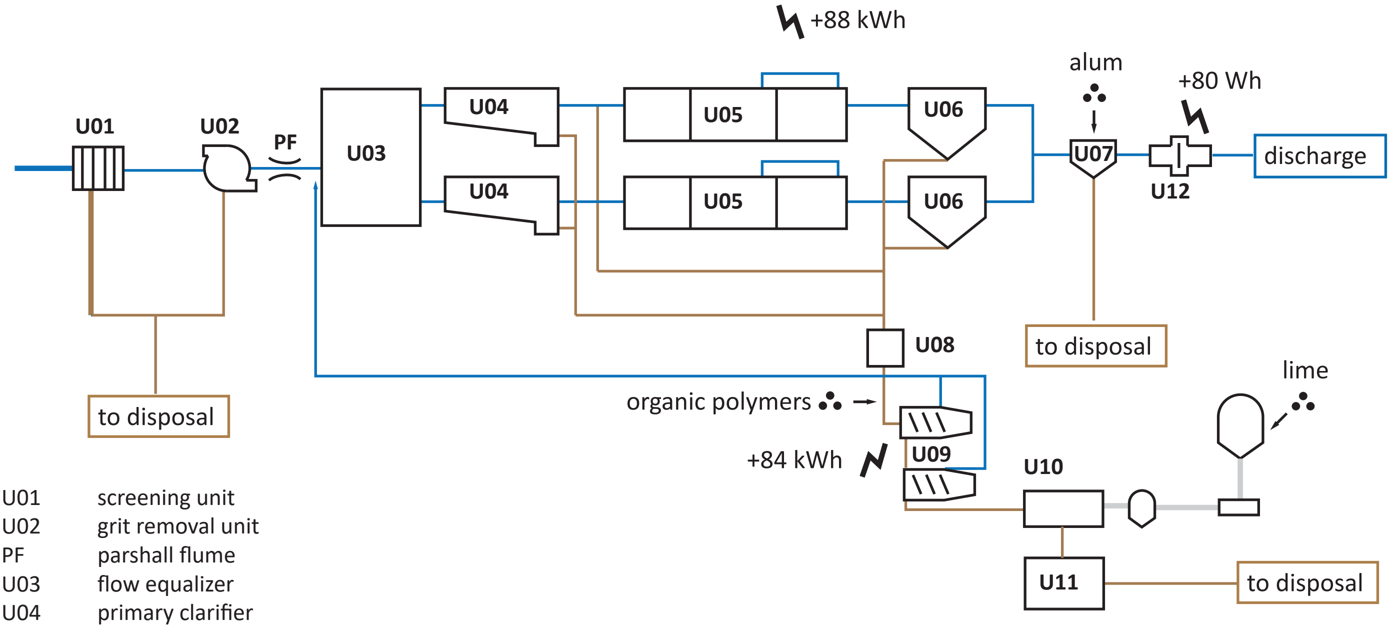
The wastewater characteristics were estimated via sampling and laboratory analysis. BOD, suspended solids, total nitrogen and phosphorus concentration were estimated at 305 mg/L,236 mg/L, 40 mg/L, 14 mg/L respectively. Effluent target requirements include BOD < 25 mg/L, total suspended solids < 150 mg/L, total nitrogen < 15 mg/L and phosphorus < 2 mg/L.
A summary of water characteristics after each process unit is shown in table 0.

The proposed system for biological treatment is an Anaerobic / Anoxic / Oxic (AAO) process that allows for standard treatment and nutrient removal. Nitrogen concentration is lowered within target levels. Phosphorus removal is completed by chemical precipitation. Effluent water is disinfected by UV light. The final effluent for the treatment plant are expected to be within requirements: BOD 18mg/L, TSS 25 mg/L, TKN < 0.5 mg/L, P <2 mg/L and coliforms 1.2 10^{-5}MPN/100 ml.

Sludge is treated by alkaline addition with the possibility of agricultural reuse. Sludge production from primary and secondary treatment is estimated at 257 kg/day. Sludge from phosphorus precipitation by alum is estimated at 400 L/day. An overview of the treatment process is given in Figure 0.

An evaluation of possible sources of green house gases was conducted. The plant will emit directly N2O and CH4 during the different steps of the process. Indirect emissions will come mainly from electrical energy used, transport and chemical used. The carbon footprint estimated emissions were not calculated but reference values were gathered.

Operational costs were determined using a cost function and estimates found in literature. The expenditure reaches 230,000 £/year. The breakdown of the costs was estimated as follows: staff 37%, energy 20%, maintenance 16%, chemical reagents 15% waste disposal 12%.



U01	screening unit
U02	grit removal unit
PF	parshall flume
U03	flow equalizer
U04	primary clarifier
U05	AAO anearobic, anoxic, oxic unit
U06	secondary clarifier
U07	Phosphorus chemical removal
U08	UV disinfection unit
U09	dewatering and thickening centrifuges
U10	alkaline stabilization unit (mixing tank, doser, storage)
U11	treated sludge storage
U12	disinfection unit (UV light)

⚡ significant electricity usage per day (pumping not included

∴ addition of chemicals

Figure 0: treatment system diagram

Table 0: summary of water quality, flow and by products for treatment system

Unit	Description	Flow m^3/h	Leght h	BODe mg/L	TSS mg/L	TKN mg/l	P mg/L	Byproducts per day	coliforms[a] MPN/100 ml
	wastewater	11-309		305	236	40	14.6		$1.2\ 10^{7}$
U01	coarse screening	11-309		274	200			155 L	$1.14\ 10^{7}$
U01	fine screening	11-309						233 L	$9.6\ 10^{6}$
U02	grit chamber	129						67 L	$7.7\ 10^{6}$
U03	flow equalizer	129	0-2						
U04	primary clarifier	129	2	183	90			341 kg	$3.8\ 10^{6}$
U05A	anaerobic tank	129	1				6.8		
U05B	anoxic tank	129	1.7			0.5[b]		323 kg	$1.55\ 10^{5}$
U05C	oxic tank	129	12	56	46				
U06	secondary clarifier	129	2	18	25			64 kg	$7.7\ 10^{4}$
U07	Phosphorus chemical removal	129		18	25		2	400 L	
U13	disinfection	129		18	25	0.5[b]	2		$1.2\ 10^{-5}$

a : estimate based on literature (Metcalf 2003: Table 12-2)

b: as NH4-N

1. Assumptions

Compared to the conceptual proposal, additional estimates and assumptions were made. Below a brief summary of the changes.

Demographic estimates. This study uses a 20-year timeframe. This is a typical return on investment period. An estimated population for 2038 is 27,310. This number is obtained by linearly projecting the growth rate in the period 2001 to 2011. The result is a 10-year growth of nearly 1%. Considering the estimated growth for the whole Durham County (ONS 2018) in the period 2016-2026 is 3%, this seems a reasonable value. (Calculation 1.1)

Sewerage system assumption. Assuming the recent construction of the township, a separate sewerage system is assumed. Direct storm water contribution is not considered. Infiltration through system leakages could be estimated at 0.5 m3/ha*day (Metcalf & Eddy 2003). Considering Newtown Aycliffe urban area (approximately 526 hectares) the entity of this contribution is not significant (8%) and was not taken into account. (Calculation 1.3)

Water quantity. A discharge estimate for the coming two decades was calculated. For 2011 the current daily consumption of 150L/person was assumed, while for 2038 ,aligned with DEFRA targets, a consumption of 130L/person was assumed. A visualization of the trend is shown in Figure 1. Despite population growth, wastewater discharge should decrease in the coming years. For this study the discharge from 2011 was used. Additionally, it is assumed that the wastewater plant will serve only the residential part of the town (Calculations 1.2)**.**
An hypotetical daily flow discharge pattern was assumed with a peak flow factor of 2.5 (Figure 2). An equalization basin was added to the process. This will provide a stable flow through the day and decrease loading shocks. The additional costs for its construction will be reduced by size savings obtained by avoiding peak flow as the design benchmark.

Water quality. A few additional assumptions were necessary for nutrient removal calculations. Total nitrogen was estimated at 40 mg/L. Estimates based on literature of COD (610 mg/L), bCOD (500 mg/L) and rbCOD (170 mg/L) were necessary for the calculation of Phosphorus removal. A summary of water characteristics is shown in Table 2.

Table 1: discharge flows

Parameter	Value
Daily average flow	3,107
Hourly average flow	129
Hourly peak flow	323

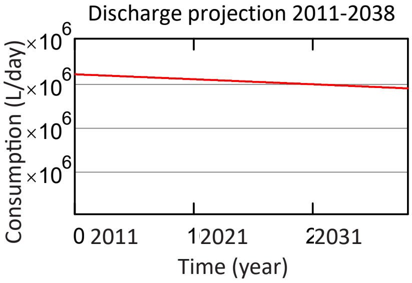


Figure 1: consumption forecast 2019-2038

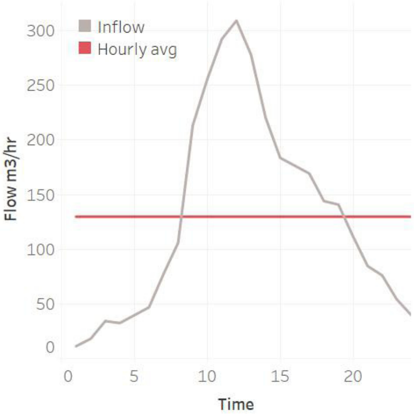


Figure 2: daily water discharge pattern

The figure shows an hypotetical inflow pattern (Inflow) used to calculate the equalization basin (Figure 3).

Table 2: water characteristics and targets

Variable	Influent	Target	Comment
BOD (mg/L)	305	25b	
COD (mg/L)	610		Estimated as BOD x 2
bCOD	500		Assumed as 1.64 of BOD[c]
rbCOD	170		Assumed as 0.3 of bCOD
Total Suspended Solids (mg/L)	236	150 b	
Volatile Suspended Solids	189		Assumed as x0.8 of TSS
pH	7.05		
NH4+ (mg N/L)	26.2		
Total nitrogen (mg/L)	40	15 b	Assumed[a]
NO3_N	7		
NOx	25		Assumed
Total P (mg P/L)	14.4	2 b	
Faecal coliforms (MPN / 100 mL)	1.2×10^7	<1000	
Helminths (eggs/L)	Nd	<1	

a: (Metcalf & Eddy 2003:213)
b: The Urban Waste Water Treatment (England and Wales) Regulations 1994
c: (Metcalf & Eddy 2003:670)

Technology proposed. Different technologies were re-evaluated using five criteria. Two related to the efficacy of the treatment: the ability to reach standard effluent quality and the possibility of incorporating nutrient removal units. A third parameter is related to the possibility and ease of numerical modelling. The increasing need for energy efficient technologies and the area footprint were the last two parameters considered. A summary of the evaluation is shown in Table 3.
Some technologies were not considered for poor energy afficiency (MBR, SBR, MBBR). Trickling filter would need additional units in the process to ensure proper nutrient removal. These additions would neutralize its advantages. Aerobic Granular Sludge in recent years proved to be an effective and interesting technology but its modelling, based on biofilm biochemistry, could not be done precisely without the involvement of an industry partner.
The technology proposed is still activated sludge but in the form of Anaerobic-Anoxic-Oxic (A/A/O) system. This setting seems to give the best performance in the parameters evaluated. A qualitative assessment of different technologies is shown in Table 3.

Disinfection. The process proposed remained UV light. This technology was considered because of its efficacy, lack of residuals in the discharge and suitability to work with low volume of water.

Sludge management. The proposed treatment changed from the initial report. Different treatments were assessed based on five parameters (Table 4): suitability to low sludge volume (C1), proximity of residential area (C2), possibility of sludge reuse (C3), energy efficiency (C4), efficiency of treatment versus costs (C5).
The proposed system includes centrifuges for thiskening and dewatering and chemical treatment using lime.
Since the sludge is expected to be reused for agricultural purposes, discharge from alum precipitation in the final stage of phosphorus removal is kept separated from the rest.

Table 3: technnology comparison for water treatment

	MBR	SBR	AGST	MBBR	TF	AAO
Effluent	3	3	3	3	3	3
Nutrient removal	3	2	3	3	1	3
Numerical modelling	2	3	2	2	3	3
Energy efficiency	1	1	2	1	3	2
Area	3	3	3	3	2	3
	12	12	13	12	12	14

Rating system:
3 : very favourable
2: favourable
1: not very favourable

MBR	membrane bio reactor
SBR	sequence batch reactor
AGST	aerobic granular sludge
MBBR	moving beds bio reactor
TF	trickling filter
AAO	aerobic, anoxic, oxic

Table 4: technnology comparison for sludge treatment

	C1	C2	C3	C4	C5
Thickening and dewatering					
Sedimentation	2	2	2	3	2
Centrifuges	3	3	3	2	3
DAF	2	2	2	1	2
Belt	2	2	3	2	3
Thermal	2	2	3	1	2
Sludge treatment					
Composting	1	1	3	2	2
Anaerobic	1	2	2	3	2
Chemical	3	3	2	2	2

Rating system:
3 : very favourable
2: favourable
1: not very favourable

Criteria:
C1: suitability to low sludge volume
C2: proximity of residential area
C3: possibility of sludge reuse
C4: energy efficiency
C5: efficiency of treatment versus costs (CAPEX+OPEX)

Assumptions: calculations.

1.1 Population projection. An estimated population for 2038 is calculated projecting the growth rate for the period 2001 to 2011.

$$P_{2001} := 26385 \quad P_{2011} := 26633 \qquad P_{2019} := 26758$$

$$dP_{10yrs} := 1 + \frac{P_{2011} - P_{2001}}{P_{2001}} = 1.009 \quad dP_{yr} := \left(dP_{10yrs}\right)^{0.1} = 1.001 \quad P_{2038} := P_{2011} \cdot dP_{yr}^{\ 27} = 2.731 \times 10^{4}$$

1.2 Discharge from population: population in 2038 is assumed 27,310. A daily consumption 150 L/day*person is assumed but DEFRA plan is to reduce water consumption to 130L/person by 2030. A return factor of 80% is used. Peak factor estimated at 2.5

$$Q_{p\_day} := 150 \frac{L}{day} \qquad f_r := 0.8 \qquad f_p := 2.5$$

Discharge flow are estimated for the next 20 years considering population change and reduced water consumption. Yearly consumption decrease is estimated at 0.66L/p.e every year considering reduction from 150L/p.e.*day to 130 in 30 years time.

$$Z(t) := P_{2011} \cdot \left(dP_{yr}\right)^{t} \cdot f_r \cdot (150 - 0.66 \cdot t)$$

Discharge 2019 $Z(8) = 3.107 \times 10^{6}$

Discharge 2029 $Z(18) = 2.993 \times 10^{6}$

Discharge 2038 $Z(28) = 2.877 \times 10^{6}$

Considering the decreasing trend, 2019 is used as a benchmark for the calculations.

$$Q_{day\_avg} := Z(8) \cdot \frac{L}{day} = 3.107 \times 10^{3} \cdot \frac{m^3}{day} \qquad Q_{h\_peak} := Q_{day\_avg} \cdot f_p = 323.607 \cdot \frac{m^3}{hr}$$

1.3 Infiltration from precipitation: An infiltration rate via system leakages of 0.5 m3/ha*day is used. This value is taken from the range given by Metcalf of 0.2-28 m3/ha*day. Infiltration is estimated as 8% of inflow and therefore not considered.

Catchment area $A_{catch} := 526 hectare$

Infiltration rate $Inf_{rate} := 0.5 \frac{m^3}{day \cdot hectare}$

Average day infiltration $Inf_{day} := A_{catch} \cdot Inf_{rate} = 263 \cdot \frac{m^3}{day}$

Infiltration compared to daily average flow: $Inf_{impact} := \frac{Inf_{day}}{Q_{day\_avg}} = 8.466 \cdot \%$

Infiltration not considered.

2. Primary treatment

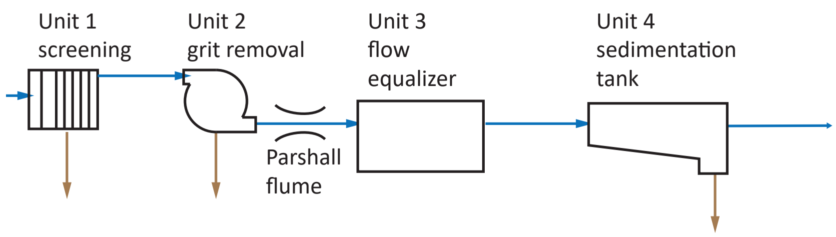


Figure 4: schematic diagram of the pretreatment phase.

Unit 1: screening (Calculation 2.1 and 2.2)

The screening cell consists of a trash rack with 80mm spaced with semi-circular sections (upstream face) and a mechanical cleaned fine screen with 13mm spacing. Both screens are inclined at 60 degrees. Even if the screening quantity is very variable depending on location, climate, topography a general estimate can be done using the following rate of grit per volume of influent: 0.004 L/m^3 (ASCE 1998:9–13). A summary of quantities and characteristics of removal is shown in Table 5: coarse screening and grit composition and estimated removed quantity. The disposal of the coarse screening should be done by delivering it to the nearest disposal site.

Unit 2: grit removal

The second unit consist of a vortex grit removal system. This solution is preferred to others for its efficiency both in term of performance (up to 90% removal) and energy. It can work on a wide flow variation and produces minimal head loss and space requirement. After removal the grit should be washed. A first estimate of measurements based on literature (Metcalf, 2003) is shown in Table 6. Grit should be disposed to handling facilities.

Odour control for screening and grit.

The pre-treatment phase may produce obnoxious odours. This is caused by emission of different gases such as hydrogen sulphide, indoles, skatoles, amines, ammonia, carbon dioxide and methane (Water Environmental Federation 1995). Some of the key variables are sludge sulphur composition, management and loading and therefore hard to predict on theoretical basis. In case this problem will arise, the management company may use one of the many available remediation which include addition of chlorine or peroxide, pH adjustment, pre-areation or air scrubbing.

Unit 3: flow equalizer (Table 8).

Considering that the peak flow factor for this community is quite high (estimated in the concept reportat 2.5), an equalization tank might be necessary. A tank of 1000 m^3 would fully equalize the flow at 129 m^3/h. This option seemed more favourable than a 790 m^3 option that would contain the peak flow only within 1.5.
Full calculations shown in Talbe 8.

Unit 4: sedimentation tank (Calculation 2.3).

Two rectangular sedimentation tanks are proposed. Given the physical and chemical composition of the sludge, the general law of sedimentation is not applicable. Overflow rate used for the design is 30 m3/m2*d. This value is just below the optimal range of of 32-48m3/m2*d (Metcalf & Eddy 2003) but resulting in an ideal 2h retention time.
The size of each sedimentation tank is 5m x 10m with a depth of 2.5m . The average retention time is 2 hours. Horizontal velocity is below organic and grit scour velocity therefore it should not create re-suspention. An estimation of BOD and SS removal was estimated giving values 33% and 54% respectively. A summary of the sedimentation unit is shown in Table 7.

Table 5: primary treatment summary

Coarse screening	
width	0.55m
bar section	semicircular 10x30mm
spacing	80mm
head loss	0.5 m
screening	155 L/day
Inclination	60 degrees
Fine screening	
screening size	13mm
removal type	mechanical
head loss	0.017m
screening	232 L/day

Table 6: grit unit summary

diameter upper chamber	4m
diameter lower chamber	1.2m
height	3m
mesh size	0.30 mm
removal rate	95%
grit removed	0.06 m^3 / day

Table 7: sedimentation tank summary

size (w,L,d)	5x10x2.5m
number of tanks	2
BOD removal	33%
TSS removal	54%

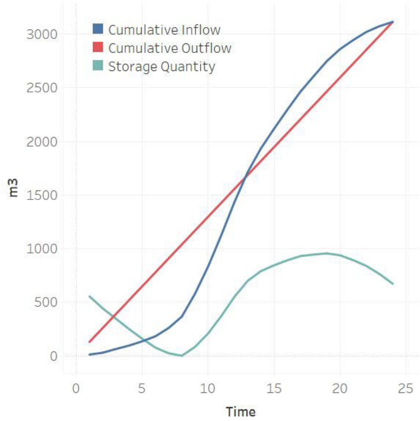


Figure 3: cumulative volume curve
The graph shows the inflow (cumulative inflow) the equalized discharge *Cumulative outflow) at an average of 129 m3/h and the volume of water stored in the tank at every hour (Storage quantity). The storage volume is estimated at 1000 m^3. Calculations shown in table 8

2.1 Pretreatment (PT) velocity considerations. Ideal flow velocity passing through a screen should be in the range of 0.4-0.9 m/s with a possible peak flow design around 0.76 m/s (ASCE 1998).

$$vel_{PT\_min} := 0.4\frac{m}{s} \qquad vel_{PT\_peak} := 0.76\frac{m}{s} \qquad vel_{PT\_max} := 0.9\frac{m}{s}$$

$$A_{PT\_min} := \frac{Q_{day\_avg}}{vel_{PT\_min}} = 0.09\,m^2 \qquad A_{PT\_peak} := \frac{Q_{h\_peak}}{vel_{PT\_peak}} = 0.118\,m^2$$

Considering a width of 0.48m the depth variation is of few centimeters only.

$$w_{PT} := 0.48m \quad h_{PT\_min} := \frac{A_{PT\_min}}{w_{PT}} = 0.187\,m \quad h_{PT\_max} := \frac{A_{PT\_peak}}{w_{PT}} = 0.246\,m$$

Pretreatment screening design. Coarse screening. the screening phase consists of a trash rack with 80mm spaced, semicircular sections (upstream face) ,bar shape factor at 1.83 (B.PT). Both screen inclined at 60 degrees.

Trash rack head loss calculation. Number of bars $n_{PT} := \frac{w_{PT}}{0.08m} + 1 = 7$ $\quad \theta_{PT} := 60deg \quad B_{PT} := 1.83\frac{s^2}{m}$

$$w_{PT\_chamber} := w_{PT} + n_{PT} \cdot 0.01m = 0.55\,m$$

$$w\_b_{PT\_ratio} := 1 \qquad H_{PT} := \frac{{vel_{PT\_peak}}^2}{2} \cdot w\_b_{PT\_ratio} \cdot \sin(\theta_{PT}) \cdot B_{PT} = 0.458\,m$$

Screening volume calculated using 50L/1000m3 (Metcalf, 2003).

$$Screen_{PT\_coarse} := Q_{day\_avg} \cdot \frac{50L}{1000m^3} = 155.331 \cdot \frac{L}{day}$$

2.2 Pretreatment design. Fine screening. Mechanical cleaned fine screen with 13mm spacing, screening removal estimated at 75L/1000m3 (Metcalf, 2003).

$$Screen_{PT\_fine} := \frac{75}{1000} \frac{L}{(m)^3} \cdot Q_{day\_avg} = 232.997 \cdot \frac{L}{day}$$

$$C_{PT\_fine} := 0.6 \qquad A_{PT\_fine} := w_{PT\_chamber} \cdot h_{PT\_min} \qquad h\_loss := \frac{1}{2g} \cdot \left(\frac{Q_{day\_avg}}{C_{PT\_fine} \cdot A_{PT\_fine}} \right)^2 = 0.017 \cdot m$$

2.3 Sedimentation tank (ST). Two rectangular tanks with a depth of 2.5m are proposed. Overflow rate range considered is 32-48 m3/m2*d (Metcalf 1991). Within this flow rate removal of SS should be between 65%, BOD removal 45% (Metcalf 1991). Equalization tank will avoid peak flows.

$$h_{ST} := 2.5m \qquad A_{ST} := \frac{Q_{day\_avg}}{2 \cdot 30 \frac{m^3}{m^2 \cdot day}} = 51.777\,m^2$$

Dimension of each tank L=10m W=5m
h=2.5m

Sedimentation tank solid retention time (HRT.ST). A typical retention time is considered to be 2.0 hr (Metcalf 1991).

$$\theta_{ST} := \frac{(A_{ST} \cdot h_{ST} \cdot 2)}{Q_{day\_avg}} = 2 \cdot hr$$

1.9 Sedimentation tank velocity considerations. To verify that settled material will not become resuspended, the horizontal velocity (vel.hor) should be slower than scour velocity. For scour veloctiy the following parameters are used:

constant for sticky material	$\beta_{ST} := 0.06$	organic matter density	$\rho_{org} := 1.2$
Darcy Weisbach factor	$f_{ST} := 0.03$	Sand silt clay density	$\rho_s := 2.65$
drag coefficient	$C_D := 10$	water density	$\rho := 1$
		particle diameter	$D_{part\_ST} := 0.02\text{cm}$

$$vel_{ST\_scour\_grit} := \left[\frac{8\cdot\beta_{ST}\cdot(\rho_s - 1)g\cdot D_{part\_ST}}{f_{ST}}\right]^{0.5} = 0.228\frac{m}{s}$$

$$vel_{ST\_scour\_org} := \left[\frac{8\cdot\beta_{ST}\cdot(\rho_{org} - 1)g\cdot D_{part\_ST}}{f_{ST}}\right]^{0.5} = 0.079\frac{m}{s}$$

$$vel_{ST\_hor} := \frac{Q_{day\_avg}}{A_{ST}} = 6.944\times 10^{-4}\frac{m}{s} \qquad vel_{ST\_hor\_peak} := \frac{Q_{h\_peak}}{A_{ST}} = 1.736\times 10^{-3}\frac{m}{s}$$

Horizontal velocities during average and epak flow are slower than both organic and grit scour velocity, no resuspencion should accour.

1.10 Sedimentation tank BOD and TSS removal. Literature (Metcalf 1991) proposes an empirical formula for BOD and TSS removal at T=20C. Considering that the average temperature is 15.72C, the removal was calculated after adjusting the retention time with a multiplier (M).
Retention time equivalent to 20C:

$$T_{avg} := 15.72 \qquad M_{ST} := 1.82\cdot e^{-0.03T_{avg}} = 1.136 \qquad \theta_{ST\_T20\_avg} := \frac{\theta_{ST}}{M_{ST}} = 1.761\cdot hr$$

BOD removal: $a_{ST\_BOD} := 0.018\cdot h\imath$ $b_{ST\_BOD} := 0.02$

$$BOD_{ST\_rem\_avg} := \frac{\theta_{ST\_T20\_avg}}{a_{ST\_BOD} + b_{ST\_BOD}\cdot\theta_{ST\_T20\_avg}}\cdot\frac{1}{100} = 33.089\cdot\%$$

BOD effluent from tank. 10% of BOD was removed in pretreatment

$$BOD_{PT\_rem} := 0.92 \qquad BOD_0 := 305\frac{mg}{L}$$

$$BOD_{ST\_eff} := BOD_0\cdot BOD_{PT\_rem}\cdot\left(1 - BOD_{ST\_rem\_avg}\right) = 187.751\cdot\frac{mg}{L}$$

TSS removal: $a_{ST\_TSS} := 0.0075\cdot hr$ $b_{ST\_TSS} := 0.014$

$$TSS_{ST\_rem} := \frac{\theta_{ST\_T20\_avg}}{a_{ST\_TSS} + b_{ST\_TSS}\cdot\theta_{ST\_T20\_avg}}\cdot\frac{1}{100} = 54.768\cdot\%$$

TSS effluent. 15% of TSS was removed in pretreatment. $TSS_{PT\_rem} := 0.85$ $TSS_0 := 236\frac{mg}{L}$

$$TSS_{ST\_eff} := TSS_0\cdot TSS_{PT\_rem}\cdot\left(1 - TSS_{ST\_rem}\right) = 90.735\cdot\frac{mg}{L}$$

Estimated total suspended solids accumulated for each day:

$$TSS_{ST\_acc} := \left[\left(TSS_0\cdot TSS_{PT\_rem}\right) - TSS_{ST\_eff}\right]\cdot Q_{day\_avg} = 341.309\cdot\frac{kg}{day}$$

Table 8: calculation of equalization basin volume.
Hourly flow variation was assumed based on a published example (Lin 2007).

hr	f hr	Out	storage	tank	balance	In sum	out sum
1	11.23	129.73	-118.50	554	-118.50	11.23	129.73
2	18.00	129.73	-111.73	442	-230.23	29.23	259.46
3	34.20	129.73	-95.53	347	-325.76	63.43	389.19
4	32.40	129.73	-97.33	249	-423.09	95.83	518.92
5	39.60	129.73	-90.13	159	-513.22	135.43	648.65
6	46.80	129.73	-82.93	76	-596.15	182.23	778.38
7	77.36	129.73	-52.37	24	-648.51	259.60	908.11
8	105.84	129.73	-23.89	0	-672.40	365.44	1037.84
9	213.12	129.73	83.39	83	-589.01	578.56	1167.57
10	255.60	129.73	125.87	209	-463.14	834.16	1297.3
11	292.32	129.73	162.59	372	-300.55	1126.48	1427.03
12	309.10	129.73	179.37	551	-121.18	1435.58	1556.76
13	277.92	129.73	148.19	699	27.01	1713.50	1686.49
14	220.32	129.73	90.59	790	117.60	1933.82	1816.22
15	183.60	129.73	53.87	844	171.47	2117.42	1945.95
16	176.40	129.73	46.67	891	218.14	2293.82	2075.68
17	169.20	129.73	39.47	930	257.61	2463.02	2205.41
18	144.00	129.73	14.27	944	271.88	2607.02	2335.14
19	140.76	129.73	11.03	955	282.91	2747.78	2464.87
20	111.60	129.73	-18.13	937	264.78	2859.38	2594.6
21	84.60	129.73	-45.13	892	219.65	2943.98	2724.33
22	75.94	129.73	-53.79	838	165.86	3019.92	2854.06
23	54.00	129.73	-75.73	763	90.13	3073.92	2983.79
24	39.60	129.73	-90.13	672	0.00	3113.52	3113.52

hr:	day time
F hr:	hourly flow
Out:	equalized hourly flow
Storage:	hourly flow balance (f hr - Out)
tank:	volume of water in sedimentation tank
balance:	cumulative balance (balcen from previous hour + balance current hour)
In sum:	cumulative inflow
Out sum:	cumulative outflow

3. Secondary treatment

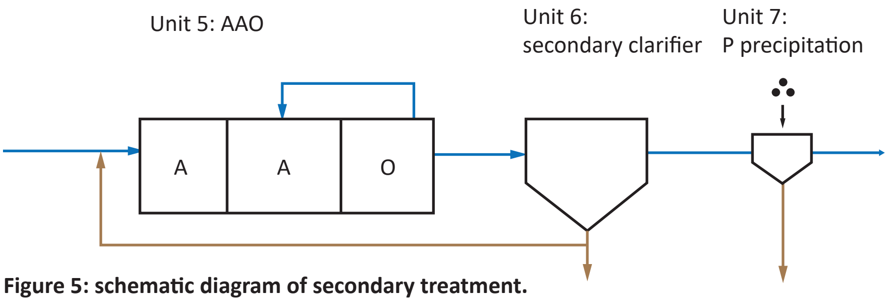


Figure 5: schematic diagram of secondary treatment.

Unit 5: Anaerobic, Anoxic Oxic unit. (Calculations 3.1 - 3.7)
The secondary treatment is aimed at nutrient and BOD removal. The Anaerobic, Anoxic, Oxic (AAO) was preferred to other nutrient removal technologies. The factors leading to this choice are: ability to reach the target nutrient removal, simplicity of construction and operation. Table 12 shows a comparison with other treatments. The AAO system, in this particular case of rich bCOD influent water, gives the most favourable solution.

Table 12: comparison between different phosphorus removal systems.

Process	N removal	P removal	SRT	Complexity	Notes
Phoredox (A/O)	no	yes	2-4 d	low	
AAO	yes	yes	7-15 d	medium	
Bardenpho	yes	yes	10-20 d	high	
Virginia Initiative Plant	yes	yes	5-10 d	high	
Johannesburgh type	yes	yes	average	high	for weak water
PhoStrip	yes	yes	5-20	high	chemical precipitation

The system was designed as two parallel lines so in case of maintenance the treatment plant can still operate.

Phosphorus removal (Calculation 3.18-3.22). The combination of the first two units (anaerobic-anoxic) provides phosphorus removal to a concentration of 6.8 mg/L. The sludge produced will contain 7% of phosphorus. Additional removal is performed after the second clarifier with the addition of liquid alum (400 L/day).

Anoxic unit (calculation 3.8-3.17). The second step of the process reduces the concentration of nitrogen. The system was designed for a 3000 MLSS concentration. This required a volume of 220 m^3 with a detention time of 1.7 hours. Alkalinity should be added to the system, estimated at 509 kg of $NaHCO_3$. Estimated oxygen requirement is 10 m^3/min. A power consumption of 88 kWh based on a 0.4 kW/m^3 consumption. Table 10 shows the main characteristics of this sub-unit.

Oxic unit. The final unit provides aerobic conditions for additional removal. Each tank has a volume of 770 m^3 with a BOD loading of 0.375 kg/m^3*day. Aeration needed was estimated at 10 m^3/min. The effluent water will contain the following concentrations: BOD mg/L, TSS 37 mg/L, NH4-N < 0.5 mg/L and P 6.8 mg/L. A summary of all the characteristcs can be seen in Table 9.

Secondary clarification. Two clarifiers with diameter of 4.8m each are proposed. A total surface of 141 m^2 should keep loading rate around 22m^3/m^2*day which belongs to the lower range of acceptable loading and therefore provide good clarification. The solid loading will reach 4.4 kg/m^2*hr. A BOD removal of 33% is estimated bringing BOD level at 18 mg/L. TSS removal is estimated at 54% bringing concentration at the effluet of the clarifier at 24 mg/L.

Table 9: summary table for aerobic tank

Average flow	3107 m^3/d
Avg BOD load	570 kg/d
Avg TKN load	124 kg/d
Aerobic SRT	9.2 days
Aeration tanks	2
Tank volume	770+770 m^3
Tank size (WxLxH)	11x17x4 (each)
HRT	12 h
MLSS	3000 g/m^3
MLVSS	2310 g/m^3
F/M	0.16
BOD loading	0.375 kg/m^3*d
Sludge production	323 kg/d
Observed yield	0.80 gTSS/g BOD
	0.64 gVSS/gBOD
Air flowrate average	10 m^3/min
RAS ratio	0.6
Clarifier application rate	22 m^3/m^2*day
Clarifier units	2
Clarifier diameter	4.8m each
Effluent BOD	57 mg/L
TSSe	37 mg/L
Effluent NH4-N	<0.5 mg/L

Table 10: summary table for anoxic design

Effluent NO_3_N	6 g/m^3
Internal recycle ratio	2.2
RAS recycle ratio	0.6
Anoxic Volume	220 m^3
MLSS	3000 g/m^3
Overall SDNR	0.273 gNO3/gMLSS*d
Detention time	1.7 h
Mixing power	88 kW
Alkalinity required	509 kg/d as $NaHCO_3$

Table 11: summary table for phosphorus removal

Effluent P	6.9 g/m^3
P in sludge	7 %
SRT	1 h
Tank volume	129 m^3

Secondary treatment calculations

3 .Nitrification parameters. Water and kinetic parameters. Parameters taken from Metcalf (Metcalf 1991 : Table8-11)

$NH4\_N_0 := 26.2\frac{mg}{L}$ $\quad TSS_0 := 236\frac{mg}{L}$ $\quad BOD_{ST\_e} := 187\frac{mg}{L}$

$DO_{min} := 2\frac{mg}{L}$ $\quad VSS_0 := TSS_0 \cdot 0.8 = 188.8 \cdot \frac{mg}{L}$

$bCOD := BOD_{ST\_e} \cdot 1.64 = 306.68 \cdot \frac{mg}{L}$ $\quad rbCOD := bCOD \cdot 0.3 = 92.004 \cdot \frac{mg}{L}$ This ratio is assumed based on example 8-2 on Metcalf

$NO_x := 23\frac{mg}{L}$ calculation at 2.3 $\quad nbVSS := 20\frac{mg}{L}$ $\quad BOD_{load} := Q_{day\_avg} \cdot BOD_{ST\_eff} = 583.273 \cdot \frac{kg}{day}$

Kinetic parameters:

$\mu_{n\_m\_T15} := 0.75\frac{gm}{gm \cdot day} \cdot (1.07)^{15-20} = 0.535 \cdot \frac{gm}{gm \cdot day}$ $\quad Y_n := 0.12$ $\quad Y := 0.4$

$k_{n\_T15} := 0.74\frac{gm}{m^3} \cdot 1.053^{15-20} = 0.572 \cdot \frac{gm}{m^3}$ $\quad k_o := 0.5\frac{gm}{m^3}$ $\quad K_s := 20\frac{gm}{m^3}$

$k_{dn\_T15} := 0.08\frac{gm}{gm \cdot day} \cdot 1.04^{15-20} = 0.066 \cdot \frac{gm}{gm \cdot day}$

$$\mu_n := \frac{\mu_{n\_m\_T15} \cdot k_o}{k_{n\_T15} + k_o} \cdot \frac{DO_{min}}{k_o + DO_{min}} - k_{dn\_T15} = 0.134 \cdot \frac{gm}{gm \cdot day}$$

3.1 Solid retention time $\quad SRT_{ntr} := \frac{1}{\mu_n} = 7.471 \cdot day$ $\quad FS := 1.3$ $\quad SRT_{design} := FS \cdot SRT_{ntr} = 9.712 \cdot day$

3.2 Biomass produced. Firstly two kinetic parameters get adjusted to 15C

$k_{d\_N\_T15} := 0.12\frac{gm}{gm \cdot day} \cdot 1.04^{15-20} = 0.099 \cdot \frac{gm}{gm \cdot day}$

$\mu_{m\_T15} := 6\frac{gm}{gm \cdot day} \cdot 1.07^{15-20} = 4.278 \cdot \frac{gm}{gm \cdot day}$

Effluent dissolved substrate

$$S_{ntr\_e} := \frac{K_s \cdot (1 + k_{d\_N\_T15} \cdot SRT_{design})}{\left[SRT_{design} \cdot (\mu_{m\_T15} - k_{d\_N\_T15})\right] - 1} = 0.989 \cdot \frac{gm}{m^3}$$

Heterotrophic biomass

$$P_{X\_het} := \frac{Q_{day\_avg} \cdot Y \cdot (bCOD - S_{ntr\_e})}{1 + k_{d\_N\_T15} \cdot SRT_{design}} = 194.014 \cdot \frac{kg}{day}$$

Cell debris

$$P_{X\_cell} := \frac{0.15 \cdot k_{d\_N\_T15} \cdot Y \cdot Q_{day\_avg} \cdot (bCOD - S_{ntr\_e}) SRT_{design}}{1 + k_{d\_N\_T15} \cdot SRT_{design}} = 27.878 \cdot \frac{kg}{day}$$

Nitrifying bacteria biomass

$$P_{X\_ntrf} := \frac{Q_{day\_avg} \cdot Y_n \cdot NO_x}{1 + k_{d\_N\_T15} \cdot SRT_{design}} = 4.379 \cdot \frac{kg}{day}$$

Nonbiodegradable VSS influent

$$P_{X\_nVSS} := Q_{day\_avg} \cdot nbVSS = 62.133 \cdot \frac{kg}{day}$$

Total biomass

$$P_{X\_bio\_tot} := \frac{P_{X\_het}}{0.85} + \frac{P_{X\_cell}}{0.85} + \frac{P_{X\_ntrf}}{0.85} + P_{X\_nVSS} = 328.334 \cdot \frac{kg}{day}$$

3.3 Amount of nitrogen oxidized to nitrate. Total nitrogen is assumed 40mg/L and N in the effluent 0.5 mg/L

$$TKN := 40\frac{mg}{L} \qquad N_{eff} := 0.5\frac{gm}{m^3} \qquad NO_x := TKN - N_{eff} - 0.12\frac{P_{X\_bio\_tot}}{Q_{day\_avg}} = 26.817\cdot\frac{mg}{L}$$

3.4 Concentration of VSS and TSS in aeration basin.

$$VSS_{aer} := P_{X\_bio\_tot} + Q_{day\_avg}\cdot nbVSS = 390.466\cdot\frac{kg}{day}$$

$$TSS_{aer} := \frac{P_{X\_het}}{0.85} + \frac{P_{X\_cell}}{0.85} + \frac{P_{X\_ntrf}}{0.85} + P_{X\_nVSS} + Q_{day\_avg}\cdot\left(TSS_0 - VSS_0\right) = 474.966\cdot\frac{kg}{day}$$

$$MLVSS_{aer} := VSS_{aer}\cdot SRT_{design} = 3.792\times10^3\ kg$$

$$MLSS_{aer} := TSS_{aer}\cdot SRT_{design} = 4.613\times10^3\ kg$$

3.5 Aeration tank volume. Volume is calculated for a 3000gMLSS/m3 concentration

$$MLSS_{aer\_conc} := 3000\frac{gm}{m^3} \qquad V_{aer} := \frac{MLSS_{aer}}{MLSS_{aer\_conc}} = 1.538\times10^3\cdot m^3 \qquad \tau_{aer} := \frac{V_{aer}}{Q_{day\_avg}} = 11.879\cdot hr$$

$$VSS_{fraction} := \frac{MLVSS_{aer}}{MLSS_{aer}} = 0.822 \qquad MLVSS_{aer\_conc} := MLSS_{aer\_conc}\cdot VSS_{fraction} = 2.466\times10^3\cdot\frac{gm}{m^3}$$

3.6 F/M ratio and BOD volumetric loading.

$$FM_{rate} := \frac{Q_{day\_avg}\cdot BOD_{ST\_eff}}{MLVSS_{aer\_conc}\cdot V_{aer}} = 0.154\cdot\frac{kg}{kg\cdot day} \qquad L_{org} := \frac{Q_{day\_avg}\cdot BOD_{ST\_eff}}{V_{aer}} = 0.379\cdot\frac{kg}{m^3\cdot day}$$

Observed yield

$$bCOD_{rem} := Q_{day\_avg}\cdot\left(bCOD - S_{ntr\_e}\right) = 949.667\cdot\frac{kg}{day}$$

$$Y_{obs\_TSS} := \frac{TSS_{aer}}{bCOD_{rem}}\cdot 1.6 = 0.\ \frac{gTSS}{gbCOD} \qquad Y_{obs\_VSS} := \frac{TSS_{aer}}{bCOD_{rem}}\cdot 0.8\cdot 1.6 = 0.64\ \frac{gVSS}{gBOD}$$

3.7 BOD effluent assume sBODe=3gm/m^3

$$TSS_{ST\_eff} = 90.735\cdot\frac{gm}{m^3} \qquad sBODe := 3\frac{gm}{m^3}$$

$$BOD_{aer\_e} := sBODe + 0.7\cdot 0.85\cdot TSS_{ST\_eff} = 56.987\cdot\frac{gm}{m^3}$$

$$TSS_{aer\_e} := BOD_{aer\_e}\cdot Y_{obs\_TSS} = 45.603\cdot\frac{gm}{m^3} \qquad asd := 21\cdot Y_{obs\_TSS} = 16.805$$

Anoxic tank

3.8 Anoxic tank. Active biomass

$$X_b := \frac{Q_{day\_avg}\cdot SRT_{design}}{V_{aer}}\cdot\frac{Y\cdot BOD_0}{1 + k_{d\_N\_T15}\cdot SRT_{design}} = 1.223\times10^3\cdot\frac{gm}{m^3}$$

3.9 Internal recycle ratio. Ratio is calculated for a concentration in the effluent of 6gm/m3

$$NO3\_N_{RAS} := 6\frac{gm}{m^3}$$

RAS ratio 0.6

$$f_{RAS} := 0.6 \qquad IR := \frac{NO_x}{NO3\_N_{RAS}} - 1 - f_{RAS} = 2.87$$

3.10 Amount of NO3 N fed into the ano

$$NOX\_N_{anox\_in} := IR\cdot Q_{day\_avg} + f_{RAS}\cdot Q_{day\_avg} = 1.078\times10^4\cdot\frac{m^3}{day}$$

$$NOX\_N_{feed} := NOX\_N_{anox\_in}\cdot NO3\_N_{RAS} = 64.672\cdot\frac{kg}{day}$$

3.11 Anoxic tank volume $\tau_{anox} := 1.7hr$ $V_{anox} := \tau_{anox} \cdot Q_{day\_avg} = 220.053 \cdot m^{3}$

F/M ratio

$$FM_{rate\_N} := \frac{Q_{day\_avg} \cdot BOD_{ST\_eff}}{V_{anox} \cdot X_{b}} = 2.168 \cdot \frac{gm}{gm \cdot day}$$

3.12 Determination of new SDNR. SDRN at 20C assumed at 0.31g/g*d

$$SDNR_{T15} := 0.31 \frac{gm}{gm \cdot day} \cdot 1.026^{(15-20)} = 0.273 \cdot \frac{gm}{gm \cdot day}$$

3.13 Nitrogen that can be reduced. The retention time in the anoxic tank has been optimized to match with the feed.

$$NO_{red} := SDNR_{T15} \cdot V_{anox} \cdot X_{b} = 73.36 \cdot \frac{kg}{day}$$ Comparison with feed $$C_{nitr} := \frac{NO_{red}}{NOX\_N_{feed}} = 1.134$$

3.14 Comparison of SDNR value with reported range of 0.04-0.42 g/g*d

$$SDNR_{MLSS} := 0.25 \cdot \frac{X_{b}}{MLSS_{aer\_conc}} = 0.102$$ value is in range

3.15 Oxygen requirement.

$$O2_{req} := Q_{day\_avg} \cdot \left(bCOD - S_{ntr\_e}\right) - 1.42P_{X\_bio\_tot} + 4.33Q_{day\_avg} \cdot NO_{x} = 35.174 \cdot \frac{kg}{hr}$$

$$O2_{credit} := 2.86 \cdot \left(NO_{x} - N_{eff}\right) \cdot Q_{day\_avg} = 9.743 \cdot \frac{kg}{hr} \qquad AOTR := O2_{req} - O2_{credit} = 25.431 \cdot \frac{kg}{hr}$$

Oxygen concentration at 15C and 94m altitude, adjustment factor of 0.989

$P_{atm} := 9.72m$ $\beta_{air} := 0.95$ $F_{air} := 0.9$ $O_{rel} := 19$ $f_{T15\_h94} := 0.989$

$P_{eff\_depth} := 4.4m$ $\alpha_{air} := 0.65$ $O2_{std} := 9.08 \frac{mg}{L}$ $DO := 2 \frac{mg}{L}$ $\rho_{O2\_T15} := \frac{0.2318}{1.226} \frac{kg}{m^{3}} = 0.189 \frac{kg}{m^{3}}$

$$O2_{T15\_h94} := 10.07 \frac{mg}{L} \cdot f_{T15\_h94} = 9.959 \cdot \frac{mg}{L}$$

$$O2_{rel} := O2_{T15\_h94} \cdot \frac{1}{2} \cdot \left(\frac{P_{atm} - P_{eff\_depth}}{P_{atm}} + \frac{O_{rel}}{21}\right) = 7.231 \cdot \frac{mg}{L}$$

$$SOTR := AOTR \cdot \left[\frac{O2_{std}}{\alpha_{air} \cdot F_{air} \cdot \left(\beta_{air} \cdot O2_{rel} - DO\right)}\right] \cdot 1.024^{15-20} \cdot \alpha_{air} \cdot F_{air} = 42.12 \cdot \frac{kg}{hr}$$

$$Air_{flow} := \frac{SOTR}{0.35 \cdot \rho_{O2\_T15}} = 10.608 \cdot \frac{m^{3}}{min}$$

3.16 Alkalinity: A pH around 7 should be maintained. Influent pH is 7.05

Alk.i is netral (pH7.05) $Alk_{used} := 7.14 \cdot NO_{x} = 191.476 \cdot \frac{gm}{m^{3}}$ $Alk_{prod} := 3.57 \cdot \left(NO_{x} - N_{eff}\right) = 93.953 \cdot \frac{gm}{m^{3}}$

$$Alk_{tot} := Alk_{prod} - Alk_{used} = -97.523 \cdot \frac{gm}{m^{3}}$$

$$Alk_{bal} := Q_{day\_avg} \cdot Alk_{tot} = -302.968 \cdot \frac{kg}{day}$$ CaCO3 or as NaHCO3 $$NaHCO3 := Alk_{bal} \cdot \frac{84}{50} = -508.986 \cdot \frac{kg}{day}$$

3.17 Power consumption . Assuming 0.4kW/m3 $Pw_{anox} := V_{anox} \cdot 0.4 \frac{kW}{m^3} = 88.021 \cdot kW$

Phosphorus removal

3.18 rbCOD available for the biological phoshorus removal. rbCOD from sedimentation tank is 90 gm/m3, rbCOD from recycle is

$bCOD_{RAS} := BOD_{aer\_e} \cdot 1.64 = 93.459 \cdot \frac{mg}{L}$ $rbCOD_{RAS} := bCOD_{RAS} \cdot 0.3 = 28.038 \cdot \frac{mg}{L}$ $rbCOD = 0.092 \frac{kg}{m^3}$

$$rbCOD_{anae} := \frac{Q_{day\_avg} \cdot rbCOD + Q_{day\_avg} \cdot f_{RAS} \cdot rbCOD_{RAS}}{\left(1 + f_{RAS}\right) Q_{day\_avg}} = 68.017 \cdot \frac{gm}{m^3}$$

$NO3\_N_0 := 0 \frac{mg}{L}$ $NO3\_N_{RAS} = 6 \cdot \frac{gm}{m^3}$

$$NO3\_N_{react} := \frac{Q_{day\_avg} \cdot NO3\_N_0 + NO3\_N_{RAS} \cdot f_{RAS} \cdot Q_{day\_avg}}{\left(1 + f_{RAS}\right) Q_{day\_avg}} = 2.25 \cdot \frac{gm}{m^3}$$

$rbCOD_{eq} := NO3\_N_{react} \cdot 6.6 = 14.85 \cdot \frac{mg}{L}$ $rbCOD_{avlb} := rbCOD_{anae} - NO3\_N_{react} = 65.767 \cdot \frac{gm}{m^3}$

$$P_{rem} := \frac{rbCOD_{avlb}}{10} = 6.577 \cdot \frac{gm}{m^3}$$

3.19 Phosphorus used for heterotrophic biomass in addition to P storage due to BPR

$Y := 0.4$ $k_d := \frac{0.12}{day}$ $k_{dn} := \frac{0.08}{day}$ $NO_x = 26.817 \cdot \frac{gm}{m^3}$ $Y_n = 0.12$ $BOD_{eff} := 20 \frac{mg}{L}$ $SRT_{AS} := 7 day$

Biomass

$$P_{x\_bio} := P_{X\_het} + P_{X\_ntrf} = 1.984 \times 10^5 \cdot \frac{gm}{day}$$

P used for biomass growth

$P_{used\_bio} := 0.015 \cdot P_{x\_bio} = 2.976 \times 10^3 \cdot \frac{gm}{day}$ $P_{used\_vol} := \frac{P_{used\_bio}}{Q_{day\_avg}} = 0.958 \cdot \frac{gm}{m^3}$

3.20 Effluent soluble P $P_{eff} := 14.4 \frac{gm}{m^3} - \left(P_{rem} + P_{used\_vol}\right) = 6.865 \cdot \frac{gm}{m^3}$

3.21 Total phosphorus in sludge: $P_{sludge} := Q_{day\_avg} \cdot \left(P_{rem} + P_{used\_vol}\right) = 23.407 \cdot \frac{kg}{day}$

$$P_{sludge\%} := \frac{P_{sludge}}{P_{X\_bio\_tot}} = 7.129 \cdot \%$$

3.22 P removal by chemical precipitation using Alum. Liquid Alum AL2(s04)3 * 18H2O with a 50% strenght and a density of 1.2 kg/L.

Weight of Aluminum per Liter: $W_{alum\_L} := 0.5 \cdot 1.2 \frac{kg}{L} \cdot 2 \cdot \frac{26.98}{666.5} = 0.049 \cdot \frac{kg}{L}$

Theoretical dosage is 1 mole of Al for 1 mol of P. $Ratio_{Al\_P} := \frac{26.98}{30.97} = 0.871$

Determining the amount of alum required per day to reduce P concentration of 6.8 mg/L to 2 mg/L. Based on literature (Metcalf 2003) 1.5 mol of Al will be needed per mole of P.

Alum per kg of P: $\text{Alum}_{kg\_P} := \text{Ratio}_{Al\_P} \cdot \dfrac{1.5}{W_{alum\_L}} = 26.901 \cdot \dfrac{L}{kg}$

Alum required per day $\text{Alum}_{day} := Q_{day\_avg} \cdot (6.8 - 2)\dfrac{mg}{L} \cdot \text{Alum}_{kg\_P} = 401.143 \cdot \dfrac{L}{day}$

Secondary clarifier

<u>3.22 Return sludge recycle ratio</u>

$X_r := 8000 \dfrac{gm}{m^3}$ $\quad X_{sludge} := 3000 \dfrac{gm}{m^3}$ $\quad \text{Rec}_{sludge} := \dfrac{X_{sludge}}{X_r - X_{sludge}} = 0.6$

<u>3.23 Secondary clarifier area.</u>

Application rate range 16-28 m3/m2*d. Assuming loading as:

$\text{Load}_{SC} := 22 \cdot \dfrac{m^3}{m^2 day}$ $\quad A_{SC} := \dfrac{Q_{day\_avg}}{\text{Load}_{SC}} = 141.21\, m^2$ $\quad \text{Dia}_{SC} := \left(\dfrac{\frac{A_{SC}}{2}}{\pi}\right)^{0.5} = 4.741\, m$

Two tanks of 4.8m diameter

<u>3.24 Secondary clarifier solid loading.</u>

$\text{SLoad}_{SC} := \dfrac{(1 + \text{Rec}_{sludge}) \cdot Q_{day\_avg} \cdot \text{MLSS}_{aer\_conc}}{A_{SC}} = 4.4 \cdot \dfrac{kg}{m^2 \cdot hr}$ (in suitable range)

<u>3.25 Secondary clarification tank BOD and TSS removal.</u> Similarly to primary sedimentation tanks, a SRT of 2h is chosen. Given a non standard temperature the SRT for calculatio nhas been adjusted.

Retention time equivalent to 20C:

$T_{avg} := 15.72$ $\quad M_{ST} := 1.82 \cdot e^{-0.03 T_{avg}} = 1.136$ $\quad \theta_{SC\_T20\_eq} := \dfrac{\theta_{ST}}{M_{ST}} = 1.761 \cdot hr$

BOD removal rate:

$a_{ST\_BOD} := 0.018 \cdot hr$ $\quad b_{ST\_BOD} := 0.02$ $\quad \text{BOD}_{SC\_rem} := \dfrac{\dfrac{\theta_{SC\_T20\_eq}}{a_{ST\_BOD} + b_{ST\_BOD} \cdot \theta_{ST\_T20\_avg}}}{100} = 33.089 \cdot \%$

$\text{BOD}_{aer\_e} = 56.987 \cdot \dfrac{mg}{L}$

BOD effluent from tank.

$\text{BOD}_{SC\_eff} := \text{BOD}_{aer\_e} \cdot \text{BOD}_{SC\_rem} = 18.857 \cdot \dfrac{mg}{L}$

TSS removal rate:

$a_{ST\_TSS} := 0.0075 \cdot hr$ $\quad b_{ST\_TSS} := 0.014$ $\quad \text{TSS}_{SC\_rem} := \dfrac{\dfrac{\theta_{ST\_T20\_avg}}{a_{ST\_TSS} + b_{ST\_TSS} \cdot \theta_{SC\_T20\_eq}}}{100} = 54.768 \cdot \%$

$\text{TSS}_{aer\_e} = 45.603 \cdot \dfrac{gm}{m^3}$ $\quad \text{TSS}_{SC\_eff} := \text{TSS}_{SC\_rem} \cdot \text{TSS}_{aer\_e} = 24.976 \cdot \dfrac{mg}{L}$

Estimated total suspended solids accumulated for each day:

$\text{TSS}_{SC\_acc} := (\text{TSS}_{aer\_e} - \text{TSS}_{SC\_eff}) \cdot Q_{day\_avg} = 64.08 \cdot \dfrac{kg}{day}$

4. Tertiary treatment

Disinfection.

The technology chosen for the final disinfection is UV light. This solution provides excellent disinfection and no residuals. Given the low water volume, operational costs are limited.

For coliforms 2-log removal a dose of 30-60 mJ/cm2 is recommended (Metcalf & Eddy 2003). The estimated coliforms at this stage of the treatment is 7.7 104 MPN/100ml (Table 4). A closed vessel UV disinfection unit is proposed. Considering hourly flow is 129 m^3/hr, an inner diameter is estimated at 300 mm based on existing products. (Evoqua 2017). An inactivation constant of 0.675 cm2/mJ was used (Ye et al. 2007). A few observations can be made at possible factors that can affect the disinfection: TSS concentration is low, the pH is neutral and this will prevent scaling. Stormwater and industrial discharge, by initial assumption, are not present in the waterflow. It can be assumed that the effluent coliform concentration will be reduced to 1.2 10^{-5} MPN/100 ml. Summary data are shown in Table 13.

Table 13: summary table for UV disinfection

Type	closed vessel
UV lamp	2
Sectional diameter	300 mm
Lamp power	80 W
Dose	33 mJ/cm^2
inactivation k	0.675 cm^2 / mJ
effluent coliform	1.2 10^{-5} MPN/100ml

Micropollutants.

Micropollutants are new set of contaminants that has been recognized worth of attention (Deblonde, Cossu-leguille, and Hartemann 2015). This has been recognized at legislative level by the EU water framework directive 2000/06/CE Annex X which contains a list of 33 substances worth of attention. One of the main characteristics of these pollutants is the yet unknown effects on human and environmental health but water plays a crucial role in their environmental fate and wastewater plants could have a key role in limiting their concentration. Even if mandatory guidelines are not in place it may be considered that, in the near future, removal of these substances will be required. In the proposed treatment system an additional membrane filtration unit could be added for such a purpose.

UV disinfection calculations

4.1 UV dose. For 2-log removal a dose of 30-60 mJ/cm2 is recommended. A close vessel of 300mm diameter is proposed. A UV unit with 3 lamps should provide enough intensity.

$$Q_{day\_avg} = 129.443 \cdot \frac{m^3}{hr} \qquad A_{uv} := (0.15m)^2 \cdot \pi = 706.858 \cdot cm^2$$

$$v_{uv} := \frac{Q_{day\_avg}}{A_{uv}} = 0.509 \frac{m}{s} \qquad t_{uv} := 0.05 \frac{m}{v_{uv}} = 0.098\,s$$

$$P_{lamp} := 80W \qquad I_{uv} := 3 \cdot \frac{P_{lamp}}{A_{uv}} = 0.34 \cdot \frac{W}{cm^2}$$

$$D_{uv} := I_{uv} \cdot t_{uv} = 0.033 \cdot \frac{J}{cm^2}$$

Influent coliform are estimated at 7.7 10^4. An inactivation constant of 0.675 cm^2/mJ is used.
The effluent coliform is within legislative limits.

$$k_{uv} := 0.675 \frac{cm^2}{10^{-3} J} \qquad N_0 := 7.7 \cdot 10^4 \qquad N_{coli} := N_0 \cdot e^{-k_{uv} \cdot t_{uv} \cdot I_{uv}} = 1.268 \times 10^{-5}$$

5. Sludge management

Sludge characteristics and quantities (calculations 5.1).
It is assumed that the wastewater source is domestic therefore it may be assumed as well that metal pollutants and trace organics are not present in significant quantity. Pathogenic organisms are present. Coliforms are present in the influent water at 1.2 107 MPN/100ml. An estimation of helminth eggs was done based on literature (Andreoli et al. 2008:45). For primary sludge 103-104 , for digested sludge 102-103 gTS. The estimated quantity of sludge from primary and secondary clarifiers is 17.8 m3/day.

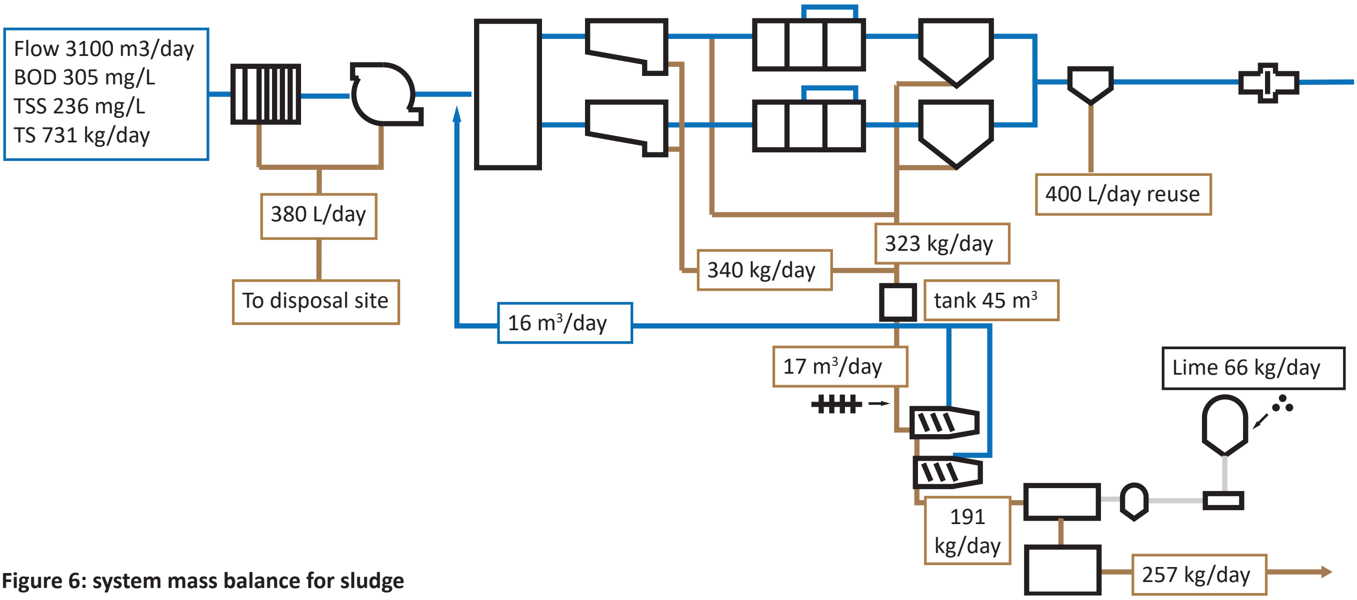


Figure 6: system mass balance for sludge

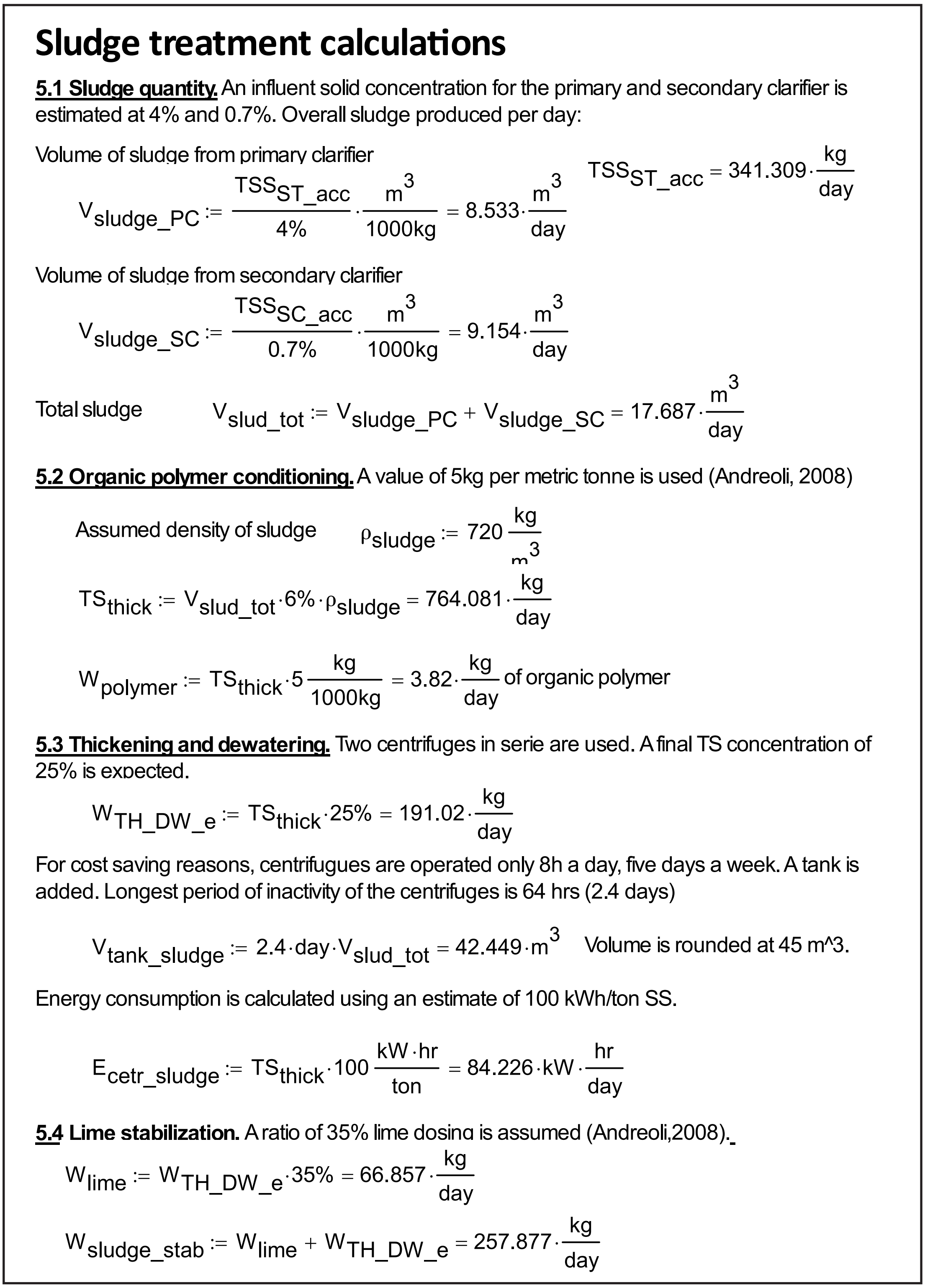
Proposed treatment (calculation 5.2 5.3 5.4).
The proposed treatment system consists of a sedimentation tank and alkaline stabilization. This solution was preferred to others for its ability to perform at lower volumes and the possibility of odour control. This last factor was weighed as very relevant considering the proximity of the township. For the thickening and dewatering phase, two centrifuges are used in sequence preceded by polymer conditioning of 5 kg/ton of sludge (calculation 5.2). This treatment should achieve a solid concentration around 25% (Andreoli 2008). Lime required for stabilization is estimated at 66 kg/day. The dosage of CaO is 30% of dry-weight. Studies (EPA et al. 2003) have shown this amount should provide an acceptable removal of pathogenic organisms. This amount will also provide a rise in pH above 12 for over 2 hours, in accordance to national guidance (DEFRA 2018). The total amount of treated sludge produced per day is estimated around 257 kg. A storage unit will be necessary before the treated sludge is sold for reuse. The suggested applications are agricultural reuse and liming agent.
Summaries of the system mass balance and treatment are shown in Figure 6 and table 14.

Sludge from chemical removal of phosphorus.
For full removal of Phosphorus, chemical precipitation is required (treatment Unit 07). Alum will be used as the chemical agent (Calculation 3.22). The sludge produced is kept separated from the overall system for a more targeted disposal and reuse. Different possibilities are available that range from aluminium recovering to agricultural use. There are still ongoing researches regarding the possible toxicity of alum in soils so further research should be done (Dassanayake et al. 2015). A precise alum sludge management plan should be defined especially in synergy with the available system of industries and facilities in the area.

Table 14: summary of disposal for treatment byproducts

Type	Quantity	Solid Conc.[a]	Volume	Disposal
Screening	380 L/day		380 L/day	sent to nearby disposal site
Grit	7.8 m^3/day	0.06 m^3/day	7.8 m^3/day	sent to nearby disposal site
Primary clarifier	341 kg/day	4 %	8.5 m^3/day	alkaline treatment and reuse
Secondary clarifier	323 kg/day	0.7 %	9.5 m^3/day	alkaline treatment and reuse
Chemical removal of Phosphorus			400 L/day	sent to other facilities for reuse

a: based on data from (Turovskii, 2006)



Sludge treatment calculations

5.1 Sludge quantity. An influent solid concentration for the primary and secondary clarifier is estimated at 4% and 0.7%. Overall sludge produced per day:

Volume of sludge from primary clarifier

$$TSS_{ST\_acc} = 341.309 \cdot \frac{kg}{day}$$

$$V_{sludge\_PC} := \frac{TSS_{ST\_acc}}{4\%} \cdot \frac{m^3}{1000kg} = 8.533 \cdot \frac{m^3}{day}$$

Volume of sludge from secondary clarifier

$$V_{sludge\_SC} := \frac{TSS_{SC\_acc}}{0.7\%} \cdot \frac{m^3}{1000kg} = 9.154 \cdot \frac{m^3}{day}$$

Total sludge $V_{slud\_tot} := V_{sludge\_PC} + V_{sludge\_SC} = 17.687 \cdot \frac{m^3}{day}$

5.2 Organic polymer conditioning. A value of 5kg per metric tonne is used (Andreoli, 2008)

Assumed density of sludge $\rho_{sludge} := 720 \frac{kg}{m^3}$

$$TS_{thick} := V_{slud\_tot} \cdot 6\% \cdot \rho_{sludge} = 764.081 \cdot \frac{kg}{day}$$

$W_{polymer} := TS_{thick} \cdot 5 \frac{kg}{1000kg} = 3.82 \cdot \frac{kg}{day}$ of organic polymer

5.3 Thickening and dewatering. Two centrifuges in serie are used. A final TS concentration of 25% is expected.

$$W_{TH\_DW\_e} := TS_{thick} \cdot 25\% = 191.02 \cdot \frac{kg}{day}$$

For cost saving reasons, centrifugues are operated only 8h a day, five days a week. A tank is added. Longest period of inactivity of the centrifuges is 64 hrs (2.4 days)

$V_{tank\_sludge} := 2.4 \cdot day \cdot V_{slud\_tot} = 42.449 \cdot m^3$ Volume is rounded at 45 m^3.

Energy consumption is calculated using an estimate of 100 kWh/ton SS.

$$E_{cetr\_sludge} := TS_{thick} \cdot 100 \frac{kW \cdot hr}{ton} = 84.226 \cdot kW \cdot \frac{hr}{day}$$

5.4 Lime stabilization. A ratio of 35% lime dosing is assumed (Andreoli,2008).

$$W_{lime} := W_{TH\_DW\_e} \cdot 35\% = 66.857 \cdot \frac{kg}{day}$$

$$W_{sludge\_stab} := W_{lime} + W_{TH\_DW\_e} = 257.877 \cdot \frac{kg}{day}$$

6. Operational carbon footprint.

Energy consumption and technology choice.
The impact of WWTP in global warming have been highlighted by Intergovernmental Panel on Climate Change (IPCC) due to the different greenhouse gases emitted during the treatment (IPCC 2006). This is exemplified by the fact that wastewater treatment plants have to comply to the Climate Change Act that sets target of 26% emission reduction by 2020 and 80% reduction of GHG by 2050, both based on 1990 values. This tendency is in most cases in contrast with the stringent discharge requirements that not only include COD and BOD removal but also nutrients and probably in near future micropollutants. In most circumstances these two goals lead to contrasting solutions (Atkins 2008).

The design of this treatment plant is a good example of the difficulty of solving the effluent requirement-energy nexus. Eutrophication potential was considered having a larger environmental impact than the one produced by greenhouse gas emissions. Solutions addressing the first issue were given priority. Without the need for nutrients removal, a more energy efficient technology, as trickling filters, could have been chosen. The proposed technology (AAO), because of the aeration required, it is not amongst the most energy efficient but better performing than other nutrient removal systems (Table 8-17 Metcalf & Eddy 2003). Attention to reduction of energy requirement was used in the choice of other units of the treatment plant. The use of an equalization basin able to absorb daily peak flows brings a lot of advantages along the process by avoiding a design based on peak flows. This causes a reduction in volumes and loading rates. Both primary and secondary clarifiers use sedimentation as main process. In both case the design allows for low surface loading and therefore a better removal. Electrolytes are used before thickening and dewatering by centrifuges to reduce the amount of power needed. Centrifuges were preferred to a more energy efficient sedimentation tank for two reasons: limited sludge amount to be treated and the high solid concentration required by an alkaline stabilization. In addition, a sludge tank before this phase allows the centrifuges to be used only for 40 hrs a week. No thermal process is used in the stabilization of the sludge and this avoids increasing the energy intensity.

Operational carbon footprint.
Several definitions exist for carbon footprint. In this study it is referred as “the total amount of direct and indirect green house gases (GHG) caused by the WWTP within the defined system boundary” (Gustavsson and Tumlin 2013). The direct and indirect emissions for this plant are summarize in Table 16. Direct emissions of CH4 and N2O will be caused by the different treatment phases. The main source for CH4 will be the oxic sub-unit with some minor emission during sludge treatment. Based on literature a range of values could be estimated between 0.25-0.7% CH4/COD (Gustavsson and Tumlin 2013). Nitrous oxide emissions will mainly be produced by the nitrogen removal process. An estimate based on previous research could be set around a value of 1% of N2O per amount of denitrified nitrogen. Regarding indirect emissions, there have been numerous studies trying to quantify energy consumption for WWTPs. The main source is the electrical requirement to run the treatment plant. From literature a breakdown of energy demand for treatment can be down as follows: 10% pumping, 55% from treatment and 35% for sludge treatment (ESMAP 2012). Other sources will consist in the carbon footprint embedded with chemical products used and transport.
Even if a precise value for energy consumption could be given only using detailed mathematical modelling coupled with actual measurements, a few references were found in published literature that could help define a reasonable range of values. Table 15 shows four values from different studies and countries. The estimates were obtained both from software modelling (AAO, China) or measurements from actual plants. The range of values obtained is not too dissimilar ranging from 0.35 to 0.52 kW/m3 of wastewater treated.

Table 15: energy intensity for different WWTP

Type	kW/m^3
AAO, China [1]	0.35
Nutrient removal, USA [2]	0.52
Typical plant, Germany [3]	0.43
Nutrient removal, Spain [4]	0.48
^{E}stimated value for this study	0.4

Sources:
1 Estimate made using Biowin modelling (Wang et al. 2016)
2 Estimate based on 4 real plants
3 Average from 5668 plants (Tang 2014 cited in Wang et al. 2016)
3 Average of 3 WWTP estimates (Molinos-Senante et al. 2011)

Future consumption.
Considering the legislative framework previously mentioned it is not unlikely to imagine that in the future energy efficiency requirements will need to increase. A first set of optimization techniques could be implemented and are shown in Table 17. Mostly these measures are related to optimization processes and good maintenance. Other possible measures that could be implemented include anaerobic process and energy recovery from wastewater. Anaerobic digestion could be implemented as a sludge treatment process. Given the limited volume of the wastewater it would probably need to be processed together with other organic waste. This could be also done by sending the sludge no a nearby anaerobic facility. Wastewater anaerobic treatment processes in temperate climate are being studied (Lema and Suarez 2017) and it may be possible in the future to reach practical applications.
Energy recovery is another active field of research. Technologies such as microbial fuel cells, once fully developed, could help harvesting at least some of the chemical energy present in wastewater. An interesting aspect of this technology is the possibility of retrofitting existing plant.

Table 16: direct and indirect emissions for the wastewater plant

Direct emissions			
Unit	Emission source	Reference value	Source
CH4	Primary sedimentation Sludge tank Sludge thickening and dewatering Discharge of wastewater	0.25%-0.7% CH_4/COD influent	(Gustavsson & S. Tumlin)
N20	AAO sludge tank discharge of wastewater	1% of the denitrified nitrogen	(Gustavsson & S. Tumlin)
Indirect emission			
Unit	Emission source	Factor	Source
CO_2 equivalent	Electricity	0.28 kgCO2e/kWh	(BEIS 2018)
	Transport	various	(BEIS 2018)
	Reagents (Alum,Organic polymers, Lime)	various	

Table 17: measures to reduce energy consumption

Unit	Description	Energy share	Possible saving (% change from total)
Pumping		10%	
	Optimization via duty software and variable speed control		-1%
	Good maintenance of plumbing fixtures and pump casing and impeller		-3%
Treatment		55%	
	Aeration optmization using system controls, last generation diffusers		-16%
Sludge	centrifuges	35%	
Total		100%	-20%

7. Operational costing

During the preparation of this report it was not possible to find publicly available costing functions for the water sector in England. This could be accepted as a possibility considering the privatization of the water sector private. An estimate was done based on studies that have been carried in Spain. It was assumed that companies in the two settings run on a similar management set up. Energy costs and personnel were adjusted to UK standards using available datasets.

Operational expenditure (OPEX) (calculation 7.1).
An estimated was made using a costing function developed by Hernandez-Sancho (Hernandez-Sancho, Molinos-Senante, and Sala-Garrido 2011). The equation is tailored for WWTP that include nutrient removal and based on several variables: COD, nitrogen and phosphorus reduction and years of operation.

Cost breakdown into energy, staff, reagents, maintenance and waste was based on previous studies (Hernández-Sancho, Molinos-Senante, and Sala-Garrido 2010). Energy and staff costing were adjusted to UK values using published datasets (EUROSTAT 2008) and (EUROSTAT 2005). It may be observed that one of the main variables to affect this estimate is the British Pound to Euro exchange rate. In the calculation a rate of 1.14 is used but it should be noticed that in the past 10 years exchange rate fluctuated significantly reaching value as high as 1.47 (2007, 2015) and as low as 1.05 (2008). Despite these issues a conversion of the cost was possible.

The overall OPEX were estimated at 232,200 £/year. The cost per m^3 of treated water is estimated at 0.205 £/m^3 (0.233 €/m^3) which is aligned with other estimates (0.2149 €/m^3) found in literature (Molinos-senante, Hernández-sancho, and Sala-garrido 2010). Energy cost are estimated at 46,430 £/year which is coherent with the estimate of 43,610 £/year calculated using an energy consuption of 0.4 kW/m3 (Table 14) (Calculation 7.2). Staff costs were estimated at 85,900 £/year which may cover 3 full time technicians. Reagents costs are estimated at 34,820 £/year. Waste disposal is estimated at 27,860 £/year which is aligned with estimate (23,360 £/year) made using a cost or 249£ per ton of dry solid (OFWAT 2015). Maintenance costs reach 37,140 £/year. This value may be lower in the first years and should allow enough budget for running the plant efficiently.

Lastly, a comparison with the data of 22 WWTPs analysed in a previous study (Molinos-senante et al. 2010) is shown in figure 8. It may be observed that the current estimate for Newtown is slightly below the average value but within a reasonable range.

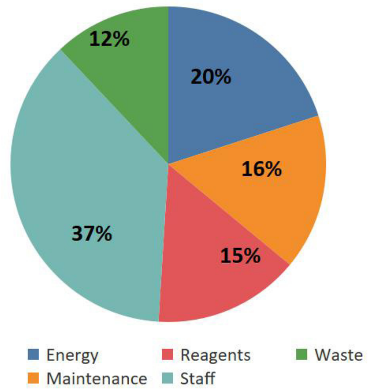


Figure 7: operational cost breakdown

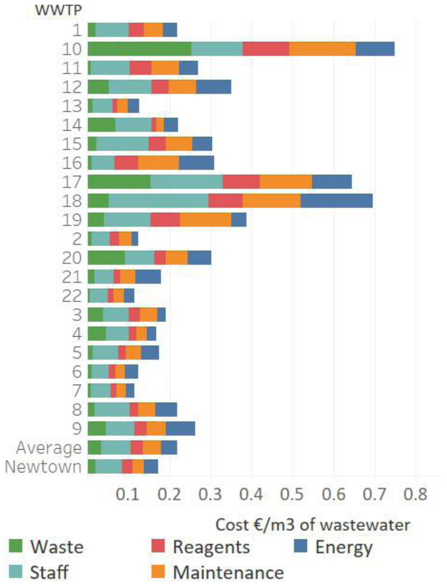


Figure 8: comparison of operational costs between Newtown WWTP and 22 WWTPs in Spain.

Operation costing calculation

7.1 Costing function.

Yearly volume of treated water $V_{yr} := 3.107 \cdot 10^{3} \cdot 365 = 1.134 \times 1(\frac{m3}{year}$

COD reduction (estimate) $COD_0 := 80\%$

Nitrogen reduction $N_0 := 75\%$

Phosphorus $P_0 := 93\%$

Years $a := 1$

$$C_{opex\_eu} := 2.518 \cdot V_{yr}^{0.7153} \cdot e^{\left(0.007 \cdot a + 1.455 \cdot COD_0 + 0.258 \cdot N_0 + 0.243 \cdot P_0\right)} = 2.647 \times 10 \frac{euro}{year}$$

$$C_{opex} := \frac{C_{opex\_eu}}{1.14} = 2.322 \times 10^{5} \frac{£}{year} \qquad C_{m3} := \frac{C_{opex}}{V_{yr}} = 0.205 \frac{£}{m3}$$

Disaggregated yearly costs was calculated using cost breakdown based on a survey (Hernandez-Sancho et al. 2010). Values were slighly changed. Original values in brackets Values in £/year

$$\mathrm{Energy} := 20\% \qquad C_{energy} := C_{opex} \cdot \mathrm{Energy} = 4.643 \times 10^{4}$$

$$\mathrm{Staff} := 37\% \qquad C_{staff} := C_{opex} \cdot \mathrm{Staff} = 8.59 \times 10^{4}$$

$$\mathrm{Reagents} := 15\% \qquad C_{reagents} := C_{opex} \cdot \mathrm{Reagents} = 3.482 \times 10^{4}$$

$$\mathrm{Waste} := 12\% \qquad C_{waste} := C_{opex} \cdot \mathrm{Waste} = 2.786 \times 10^{4}$$

$$\mathrm{Maintenance} := 16\% \qquad C_{maint} := C_{opex} \cdot \mathrm{Maintenance} = 3.714 \times 10^{4}$$

Cost adjustment. Energy and staff costing is adjusted to UK standards using EUROSTAT datasets. Energy adjustment factor calculated as the percentage difference between Spain and UK average costs for industry (EUROSTAT 2008).

$$F_{energy} := \frac{11.44}{9.87} = 1.159$$

Personnel adjustment factor calculated as the percentage different between Spain and UK average personnel cost in the water sector (EUROSTAT 2005).

$$F_{staff} := \frac{39.6}{35.4} = 1.119$$

Adjusted costs can be calculated for energy and staff

$$C_{energy\_uk} := C_{energy} \cdot F_{energy} = 5.382 \times 10^{4}$$

$$C_{staff\_uk} := C_{staff} \cdot F_{staff} = 9.609 \times 10^{4}$$

7.2 Reference values. A GBP/Euro rate of 1.14 was used.

Energy

$$E_{hr} := 0.4 \frac{\mathrm{kW} \cdot \mathrm{hr}}{\mathrm{m}^3} \cdot 129 \frac{\mathrm{m}^3}{\mathrm{hr}} = 51.6 \cdot \mathrm{kW}$$

$$E_{yr} := E_{hr} \cdot 365 \cdot 24 = 4.52 \times 10^{5} \cdot \mathrm{kW}$$

$$C_{energy\_ref} := E_{yr} \cdot \frac{0.11}{\mathrm{kW}} = 4.972 \times 10^{4}$$

Yearly electricity cost 49720 euro or 43610 £

Waste cost. Based on reported data (OFWAT 2015) a disposal cost for ton of dry solid of 249£ is used. Daily production was previously estimated at 257 kg/day.

$$C_{waste\_ref} := 257 \frac{\mathrm{kg}}{\mathrm{day}} \cdot \frac{249}{10^{3}\mathrm{kg}} \cdot 365 \cdot \mathrm{day} = 2.336 \times 10^{4} \frac{£}{\mathrm{year}}$$

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