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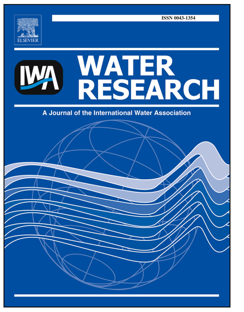

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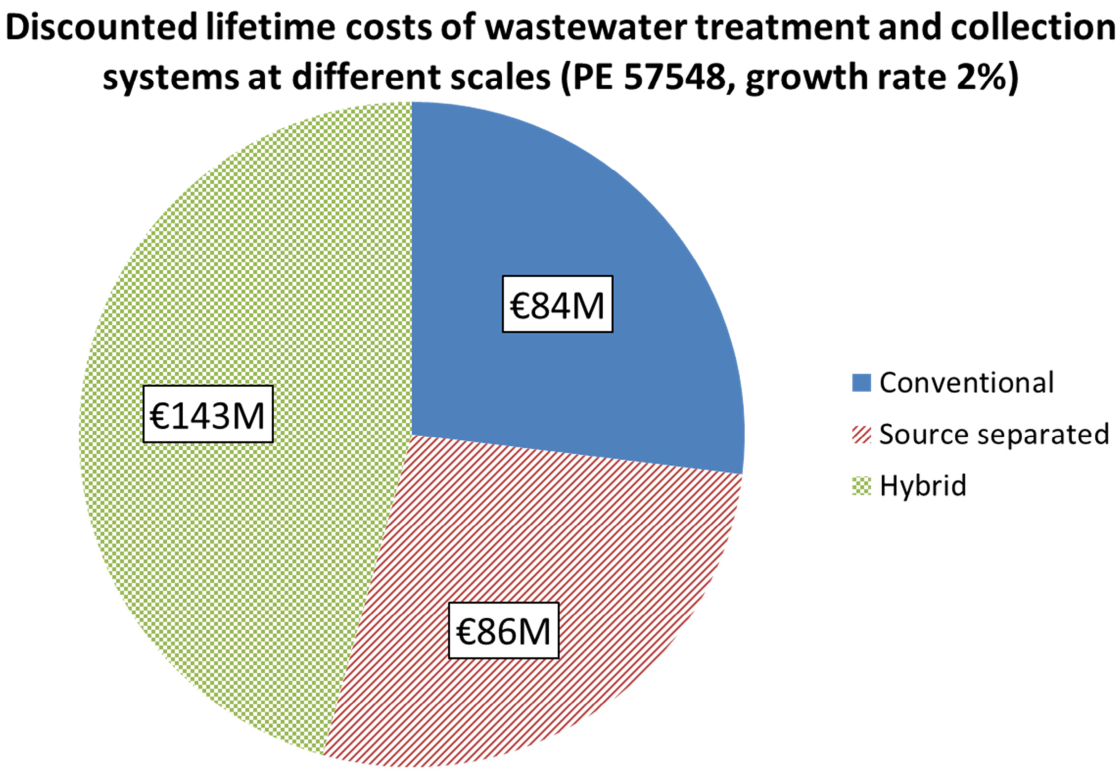
Discounted lifetime costs of wastewater treatment and collection systems at different scales (PE 57548, growth rate 2%)
€84M
€143M
€86M
Conventional
Source separated
Hybrid

Centralised, decentralised or hybrid sanitation systems? Economic evaluation under urban development uncertainty and phased expansion

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Abstract

Sanitation systems are built to be robust, that is, they are dimensioned to cope with population growth and other variability that occurs throughout their lifetime. It was recently shown that building sanitation systems in phases is more cost effective than one robust design. This phasing can take place by building small autonomous decentralised units that operate closer to the actual demand. Research has shown that variability and uncertainty in urban development does affect the cost effectiveness of this approach. Previous studies do not, however, consider the entire sanitation system from collection to treatment. The aim of this study is to assess the economic performance of three sanitation systems with different scales and systems characteristics under a variety of urban development pathways. Three systems are studied: (I) a centralised conventional activated sludge treatment, (II) a community on site source separation grey water and black water treatment and (III) a hybrid with grey water treatment at neighbourhood scale and black water treatment off site. A modelling approach is taken that combines a simulation of greenfield urban growth, a model of the wastewater collection and treatment infrastructure design properties and a model that translates design parameters into discounted asset lifetime costs. Monte Carlo simulations are used to evaluate the economic performance under uncertain development trends. Results show that the conventional system outperforms both of the other systems when total discounted lifetime costs are assessed, because it benefits from economies of scale. However, when population growth is lower than expected, the source-separated system is more cost effective, because of reduced idle capacity. The hybrid system is not competitive under any circumstance due to the costly double piping and treatment.

Keywords

Discounted life time costs, Decentralisation, Phased design, Greenfield urban expansion

List of acronyms and abbreviations

Abbreviations

BW	black water
CAPEX	capital expenditure
CAS	conventional activated sludge
DM	Dry Matter
EU-28	European Union 28 member states
GW	grey water
KW	kitchen waste
OLAND	oxygen-limited autotrophic nitrification/denitrification process
OPEX	operating expenses
UASB	upflow anaerobic sludge blanket
UWIM	urban water infrastructure model
V-SSF	vertical subsurface flow
WWTP	Wastewater Treatment Plant

Notations

λ	growth rate [%]
t	time [T]
Pt	load at time t [PE]
P0	load at time t_0 [PE]
UGt	uncertain growth at time t [PE]
Vt	variability at time t [-]
Ot	growth error at time t [-]

φt	random number at time t [-]
Q_{max}	max (or peak) wastewater discharge [m^3/h]
Q_{base}	maximum flow per household [m^3/h]
TU	flow tapping units [m^3/h]
N_{TU}	number of tapping units in the installation [-]
N	number of households connected to the sewer [-]
D	hydraulic diameter of the vacuum or pressure pipe [m]
d	day
f	Darcy friction factor [-]
L	length of pipe [m]
ΔH	friction factor [m]
Q_{full}	capacity of a filled pipe [m3/s]
v_{full}	maximum flow velocity [m/s]
n	Manning-strickler coefficient [$m^{1/3}$/s]
S	slope of the pipe [m/m]
X	treatment technology specific parameter determining the investment [-]
a, b	scaling factors empirically determined reflecting economies of scale [-]
DLC	discounted lifetime costs [€]
r	discount factor [%]
T_p	planning horizon [T]

Unit

PE	population equivalent
€	cost
T	time

hh/ha household/hectare, density

ha hectare

1 Introduction

Traditionally, sanitation systems are built to be robust, that is, they are planned so that they can cope with future variability and withstand extreme events. Sewers and drainage systems are designed for peak rainfall events and expected future wastewater discharge, while treatment systems are constructed based on predicted population growth over the asset lifetime (Rivas et al. 2008) . It has been shown that this approach to design can result in inefficient operation when predictions of future developments do not become reality. Panebianco and Pahl-Wostl (2006) describe a case where a combination of increased water efficiency and economic decline made a centralised wastewater collection and treatment system operational only with additional inputs of energy and water. Similarly, Maurer (2009) and Wang (2014) show that over capacity of treatment systems increases operational costs per customer significantly. Furthermore, building robust omits the option to implement changes over time through flexibility and phasing of designs or plans (Spiller et al. 2015).

Incorporating the idea of phased expansion into the design and plans of urban wastewater systems has been shown to be beneficial. For drainage systems, Deng et al. (2013) used the Net Present Value (NPV) and historical rainfall variability to generate a stochastic model to test a phased and non-phased design for implementing drainage systems. They concluded that by enabling expansion options, costs were reduced and the systems were prepared to respond to future developments. The studies of Marques et al. (2014) and Huang et al. (2010) for the design of water networks and Gersonius et al. (2013) for urban drainage systems, applied a scenario tree method and genetic algorithm to optimise network design under uncertainty. Using total life cycle net present value as an indicator they show that a flexible design is more cost-effective to implement than conventional robust systems. However, Marques et al. (2014) illustrate that the initial investment to enable this flexibility is 12% above an investment without flexibility. For wastewater treatment Maurer (2009) and Wang (2014) show that under uncertainty and urban growth, the idle capacity and costs of traditionally designed wastewater treatment plants are higher than those of decentralised systems, which are able to grow incrementally. Therefore, by designing in smaller units the financial risk can be reduced (Tchobanoglous and Leverenz 2013). Fane and Fane (2005) indicate that a sequence of decentralized units can be installed at a lower present value cost than a large centralized system with the equivalent Life Cycle Costs per unit of wastewater treated. Maurer (2009) gives a more cautious conclusion and suggests that a good entry market for decentralised treatment is in areas with high demand growth and large forecast uncertainty because they can respond to demand changes flexibly. Wang (2014) confirmed the findings of Maurer (2009), arguing that under most circumstances investment into decentralised treatment systems can be viable as a result of cost saving on idle capacity.

In summary of the above, phasing of infrastructure development has been shown to be cost effective by reducing idle capacity and by reducing future costs for implementing change. From the previous research, however, it becomes apparent that there is no integrated analysis of wastewater treatment and sewer systems. The studies of Maurer (2009) and Wang (2014) only consider the treatment, but do not include the collection system. An integrated analysis is however crucial, because the major share of the capital investment can be incurred by the sewer systems. Indeed, as a result of the emergence of source-separated sanitation systems (Verstraete and Vlaeminck 2011), it is likely that an even larger share of the system costs will be incurred by the sewer system as more pipes need to be installed . In source-separated systems grey water (GW – definition Table 1) and black water (BW – definition Table 1) are collected and transported in separate pipes (Zeeman et al. 2008). This is claimed to result in a number of benefits that help in progressing towards sustainability, which include:

- Water savings from highly water efficient toilet systems and vacuum pipes, which can reduce water consumption by about 25% (Zeeman et al. 2008).
- Improved recovery of energy, nitrogen and phosphorus, as the wastewater is more concentrated and wastewater streams are more homogenous. For example, BW contains about 90% of the N, 77% of the P and 55% of the COD, while it represents only about a third of the volume of municipal wastewater (Kujawa-Roeleveld and Zeeman 2006).
- Improved properties for reuse of BW sludge in agriculture, because contamination with heavy metals is low, as they are primarily from dietary sources (Tervahauta et al. 2014).

Despite these benefits, the costs of the additional pipes may pose a significant economic barrier to the transition to source-separated sanitation. However, the cost balance is not only influenced by the number of pipes, but also by their diameter and length as well as the economies of scale of the treatment. GW and BW pipes can have a smaller diameter and, because source-separated systems are mostly decentralised, a shorter pipe length, therefore potentially reducing costs. Contrary to this, decentralised source-separated systems do not benefit from economies of scale as they generally serve fewer population equivalents (PE) than the conventional centralised systems. Given these linked parameters an analysis is required that evaluates the cost balance between different treatment and collection systems. As illustrated above, such a study must include the dynamics and uncertainty of urban development as otherwise misleading lifetime costs will be determined. The aim of this study is therefore to assess the economic performance of three sanitation systems with different scales and systems characteristics under a variety of urban development pathways. In working towards this aim we provide insights into the sanitation system choice under different urban development trajectories. More specifically, we can indicate when a conventional centralised or a more decentralised source-separated sanitation system should be selected.

Table 1 Wastewater streams defined (Eriksson et al. 2002, Kujawa-Roeleveld and Zeeman 2006).

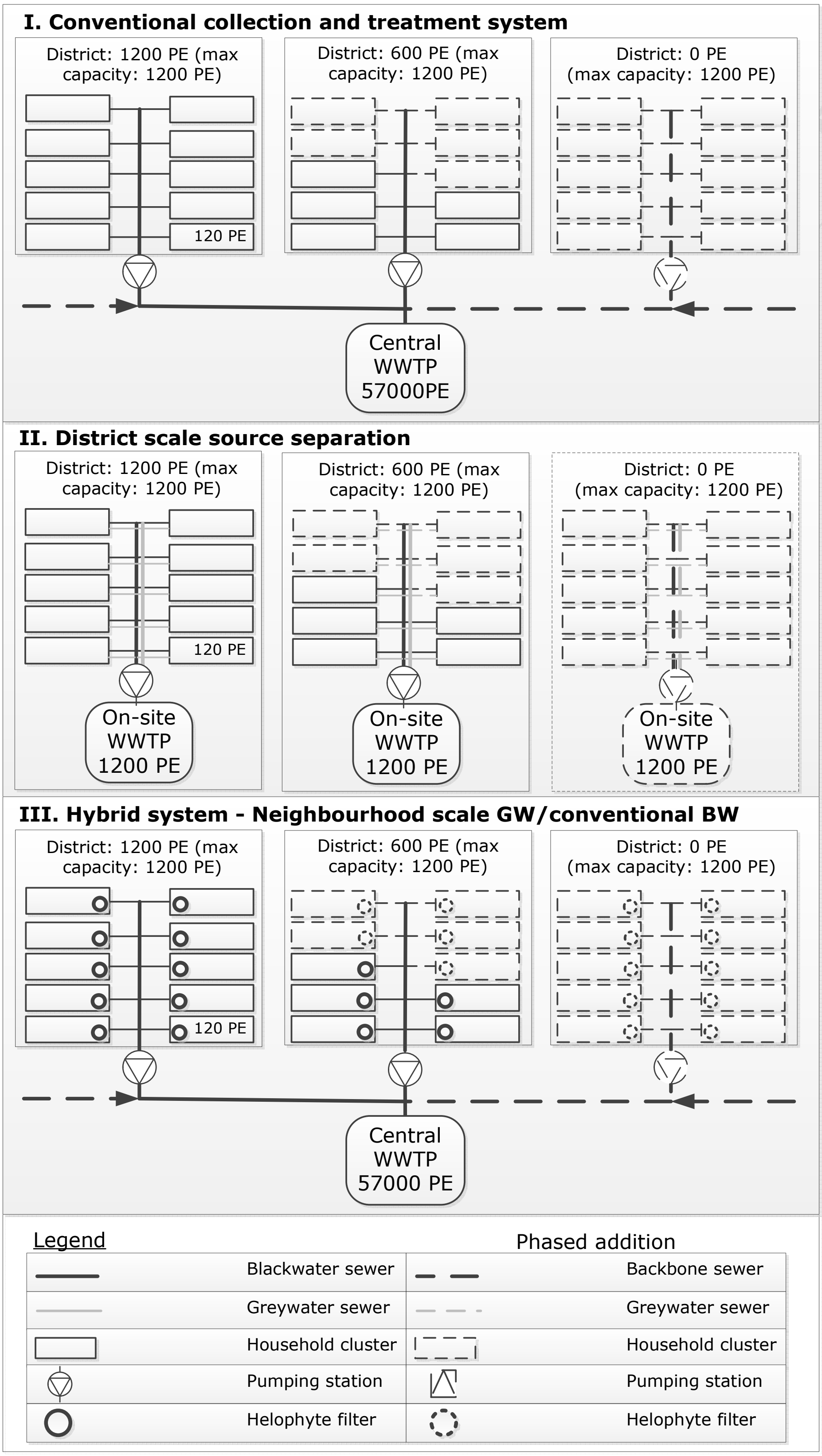
	Municipal sewage	**Grey Water (GW)**	**Black Water (BW)**	**Kitchen waste (KW)**
System	- Conventional	- Source-separated - Hybrid	- Hybrid - Source-separated	- Source-separated
Description	All wastewater collected from all household activities, but excluding rainwater.	Wastewater generated from water use activities in the kitchen, hand washing, showering and doing the laundry.	Wastewater collected from the toilet incl. urine, faeces, toilet paper and flushing water.	Organic waste such as food leftovers or peels grinded in small pieces and collected together with the black water via vacuum pipes.

Three sanitation systems were selected to work towards the set objective (Figure 1). The criteria for the system selection were that cost information is available and that the systems differ sufficiently in the scale and system characteristics. The scale and system characteristics were addressed as follows (more detail in section 2). For the conventional collection and treatment system - configuration I, municipal sewage is collected in a single pipe and treated at a Conventional Activated Sludge (CAS) treatment plant. In this system phasing can only take place in the sewerage network, but benefits from economies of scale are likely at the treatment plant. For the district source-separation system - configuration II, GW and BW+KW are collected separately and both treated at the district scale. With this system, it is evaluated whether decentralization can save costs by phased implementation of the treatment and the sewer network. In addition, the impact of costs savings from avoiding larger diameter pipes between districts and to the centralised treatment plant is evaluated. The final system is a hybrid between centralised and decentralised collection and treatment – configuration III.

Here, BW is collected separately and treated at the centralised CAS plant, while the larger volume of GW is treated in a vertical subsurface flow (V-SSF) constructed wetland (section 2.3.3) located in the neighbourhood. In other words it is a mix of the traditional way of wastewater collection and central treatment for the BW and local collection and treatment of the GW. By adopting this system configuration the economic viability of so called "hybridisation" is assessed where new solutions are added to the legacy of old systems, resulting in a gradual process of change in the water sector (Marlow et al. 2013).

1 Figure 1 Sanitation systems studied, including the phased expansion steps shown as dashed

2 lines (WWTP – Wastewater Treatment Plant).

3 

2 Methodology

The model developed in this research comprises three components. Firstly, a model of the urban growth and wastewater discharge, including the uncertainty related to it. Secondly, a model of the physical design properties of the urban area and its infrastructure. Thirdly, a model that translates design parameters into discounted asset lifetime costs.

2.1 Greenfield urban growth and wastewater discharge

For this study a fictive city based on European characteristics was designed. The city is served by a central wastewater treatment plant (WWTP) with a capacity of 70000 PE. As a consequence of population growth the WWTP is at capacity and a new wastewater collection and treatment systems needs to be implemented. The population growth and urban expansion takes place at the city fringes as a greenfield development (i.e. all infrastructure is new and laid in pristine ground), no infill urban development is considered. The urban expansion is initially modelled with a growth rate of λ = 2% over the planning horizon of the infrastructure of 31 years[1]. This time period has been selected because it represents the asset life time of wastewater treatment infrastructure (Maurer 2009). A 2% growth rate, is reflecting the average urban population growth in Europe in the period from 1990 to 2012 (UN 2012). Growth predictions for the coming 30 year are somewhat lower (about 1.3% (UN 2012). However, locally growth rates may exceed this value, as is evident in the development of "new towns" in the Netherlands (e.g. Almere 2.4% over period 2000-2015). To test the sensitivity of the results with other realised growth rates in the future, scenarios with a growth rates of 0.5%, 1.0%, 1.5% and 2.5% have been implemented. By using equation 1 the PE served at the end of the planning horizon is determined (Maurer 2009).

[1] since at t_0 construction of the sanitation system starts but finishes in t_1, the planning horizon amounts to 31 years

$$P_t = P_0 \cdot e^{\lambda \cdot t} \quad \text{Equation 1}$$

Where Pt = the load in PE at time t, P0 = PE at time 0, $\boldsymbol{\lambda}$ = growth rate and t = time.

Over the planning horizon occupation of the urban expansion may vary as a result of different real estate market developments as was for instance the consequence of the financial crisis of 2008. This variation within a general trend of a prediction is accounted for by applying a random walk to the exponential growth curves using equation 2 and 3 (Spiller et al. 2015).

$$UG_t = P_t \cdot V_t \quad \text{Equation 2}$$

$$V_t = (1 - O_t) + 2 \cdot O_t \cdot \varphi_t \quad \text{Equation 3}$$

Where UG_t = uncertain growth of PE at time t, V_t = variability at time t, O_t = growth error at time t and φ_t = random number between 0 and 1. As predictions that are further in the future are more uncertain, O_t increases over time from t_0=0.078 to uncertainty at t_{31}= 0.25. The growth error O_t and its rate of increase were chosen arbitrarily, it was assumed that a value of 0.25 is a sufficient maximum since higher uncertainties are captured with the different growth rate scenarios. For all growth rates, 1000 scenarios over the planning horizon were generated by running Monte Carlo simulation (Matlab Mathworks®), where the parameter φ_t was changed randomly at each run.

2.2 Design properties of the urban network infrastructure

The modelled population growth is used as an input to model the physical expansion of the city and its infrastructure (Figure 2). The smallest unit of expansion in this city is the neighbourhood with 50 households and 2.4 (average EU-28 in 2013 (EUROSTAT 2015) inhabitants per household or a total of 120 PE. Every neighbourhood is part of a district that comprises of 10 neighbourhoods or a total of 1200 PE. Every time a district exceeds 10 neighbourhoods a new district is initiated until it again exceeds 10 neighbourhoods or a PE of 1200.

In the conventional and hybrid systems all districts are connected to a collection system that connects district to district and to the WWTP. All neighbourhoods are themselves connected to a backbone pipe within the district (Figure 1 & Figure 2). Therefore in our growth model, every time a new district is constructed a new district backbone pipe is built, no matter whether the district is fully inhabited. All lower hierarchy pipes at neighbourhood scale are installed when the neighbourhood is built and then connected to the district backbone pipe.

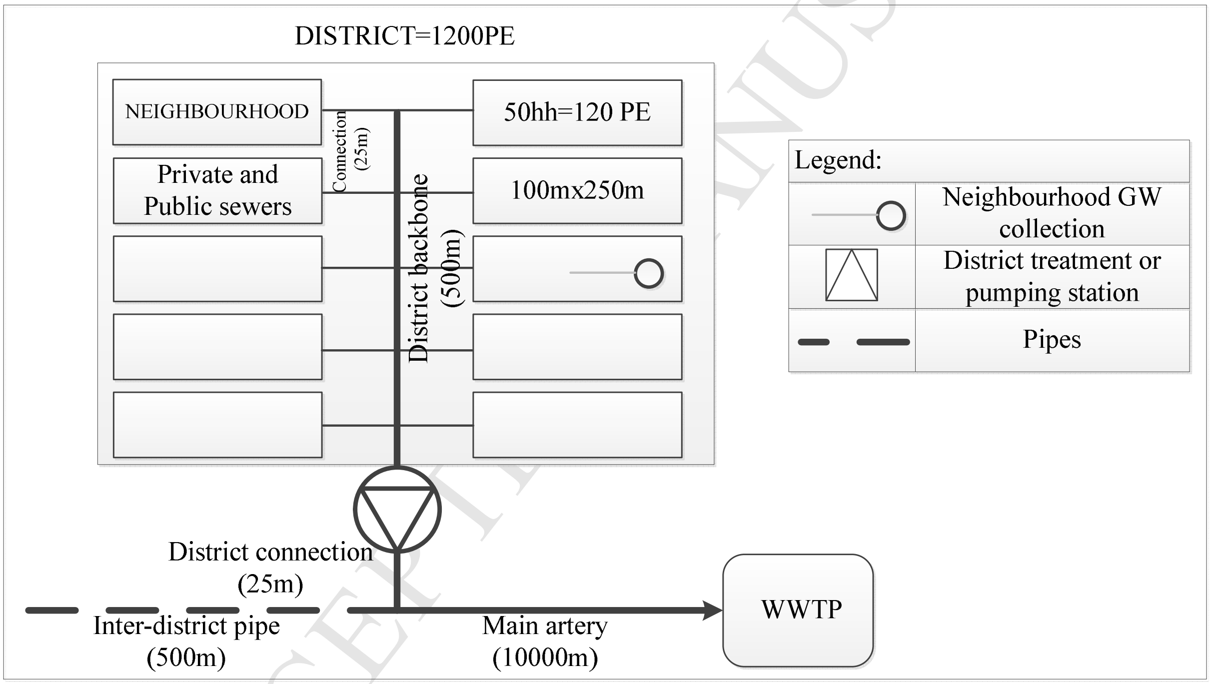


Figure 2: Layout and labels of the district and the neighbourhood.

The density of the new development is 20 hh/ha, which is a common value for residential areas with free standing or double houses in the Netherlands, Belgium and Germany (Tennekes and Harbers 2012). Since there are 50 hh/neighbourhood, each neighbourhood has a surface area of 2.5 ha. For the model it is assumed that each neighbourhood has a surface layout of 100x250 metres. The pipe length per neighbourhood is calculated by using the

Urban Water Infrastructure Model (UWIM) developed by Maurer et al. (2013). In this model pipe lengths are calculated based on an area that is arranged in an orthogonal layout (see supplementary material, section 1). The neighbourhood unit for our study has public sewers of a length of 657 m and private sewers[2] of a length of 612 m (see supplementary material, Table 1 for specifications). All sewers are separated sewers, rainwater drainage is not taken into account. Gravity sewerage is implemented up to district scale, with exception of system II, which makes use of vacuum BW sewerage. Sewers connecting districts and supplying the WWTP are pressurised as it is commonly the case in flat countries such as the Netherlands.

To determine the hydraulic diameter the Q_{max} = max (or peak) wastewater discharge must be estimated [m^3/h]. In this research the wastewater discharge is estimated from the peak water use given by Vreeburg (2007):

$$Q_{base} = TU\sqrt{N_{TU}} \qquad \text{Equation 4}$$

$$Q_{max} = Q_{base} \times \sqrt{N} \qquad \text{Equation 5}$$

Where: Q_{base} = maximum drinking water flow per household [m^3/h], TU = Tapping Units at 300 l/h, N_{TU} = Number of Tapping Units in the installation and N = Number of households connected to the sewer. The GW share of Q_{max} is 75% and BW share 25% (Al-Jayyousi 2003, Eriksson et al. 2002, Friedler and Hadari 2006). The maximum flow per household (Q_{base}) for vacuumed BW combined with kitchen waste (KW) is 9 L/p/day (Meulman unpublished). Having determined Q_{max} the pipe diameter for gravity, pressurised and vacuum sewer can be calculated. For the pressurised and vacuum system the hydraulic diameter of a sewer pipe is given by the transformed Darcy-Weisbach equation:

[2] In this study private sewers are defined as the sewers from the house until the first Y-joint that makes connection the water main (Maurer et al. 2013). This is the legal definition applied in Switzerland and the practice in the Netherlands. In the UK this type of sewer falls under the responsibility of the water companies.

$$D = \left(\frac{0.0826 \cdot f \cdot L \cdot {Q_{max}}^2}{\Delta H}\right)^{1/5} \qquad \text{Equation 6}$$

Where: D = hydraulic diameter of the vacuum or pressure pipe [m], f = Darcy friction factor at 0.02 (dimensionless and is assumed constant), L = length of pipe [m] and ΔH = the friction factor (describes the pressure drop which equals 7m for a vacuum pipe). As in practice pressure and vacuum pipes are available in 70 mm, 90 mm, 110 mm, 150 mm, 200 mm, 250 mm, 315mm 350 mm, and 400 mm diameters, the dimension diameter >D was selected.

For gravity sewer the diameter is calculated with the Manning-Strickler equation for circular section flowing full (Maurer et al. 2013) (equation 7 & 8).

$$Q_{full} = v_{full} \cdot \frac{\pi}{4} \cdot D^2 \qquad \text{Equation 7}$$

$$v_{full} = \frac{1}{n} \cdot \left(\frac{D}{4}\right)^{2/3} \cdot \sqrt{S} \qquad \text{Equation 8}$$

Where: Q_{full} = capacity of a filled pipe [m^3/s], v_{full} = max. flow velocity [m/s], D = pipe diameter [m], n = Manning-Strickler coefficient (empirically determined, 0.01 $m^{1/3}$/s), and S = slope of the pipe (set at 0.001 m/m). A pipe diameter is suitable if Q_{max} (Equation 5) is smaller than 80% of Q_{full}. Options for pipe diameters for gravity sewers are 110mm, 200mm, 300mm or 400mm. In this research we also apply the legislation and practice for pipe diameter selection as implemented in the Netherlands, where it is prescribed that public gravity sewers have a diameter of 200mm even if hydraulic diameter << pipe diameter. Similarly, private sewers are commonly PVC pipes of 110mm as the minimum (RIONED 2008).

2.3 Wastewater treatment infrastructure

Three different treatment configurations are used in this study. In table 2 a comparison is made between the effluent characteristics of the CAS systems, the effluent of a source-separated pilot installation in the Netherlands (Butkovskyi et al. 2015, STOWA 2014) and a hybrid between a vertical subsurface flow constructed wetland for GW treatment and a CAS for BW treatment. The source-separated system has lower total phosphorus (>0.2 g total phosphorus d^{-1} PE^{-1} lower) and total nitrogen loads (> 2g total Nitrogen d^{-1} PE^{-1} lower) than the conventional and the hybrid treatment (Table 2). The effluent loads for COD are comparable between source-separated and the average value for the hybrid treatment, the conventional CAS shows about $1gO_2$ per PE and day higher values. The source-separated systems further produces about 50% less sludge than a conventional system with a lower heavy metal content (Tervahauta et al. 2014).

Table 2: Comparison of effluent and sludge loads of the three treatment options studied. Values for conventional, hybrid BW and source-separated systems are yearly averages measured for the source-separation pilot installation in Sneek Noorderhoek (The Netherlands) and the wastewater treatment plant Deventer (The Netherlands) (STOWA 2014). GW values are based on 15 samples taken from 3 different vertical subsurface flow constructed wetlands in Culemborg The Netherlands (Nanninga 2011, STOWA 2014). The data range is provided for GW, as it is taken from individual samples. The values for the BW hybrid are estimated using the same performance assumptions as for the conventional treatment. For the hybrid system individual values for GW and BW are given as discharge takes place at different locations.

Effluent and sludge loads					
Parameter	Unit	Conventional	Source-separated	Hybrid	
				GW	BW
Total COD	$gO_2\ d^{-1}\ PE^{-1}$	6.7	4.9	Avg. 1.4 0.9 - 3	3.76
Total N	$g\ d^{-1}\ PE^{-1}$	2.1	0.6	Avg. 0.9 0.2 - 2.1	1.90
Total P	$g\ d^{-1} PE^{-1}$	0.3	0.1	Avg. 0.06 0.02 - 0.1	0.23
Sludge	$kgDW\ PE^{-1}\ y^{-1}$	16.7	9.2	no information	no information

2.3.1 Conventional collection and treatment (I) – phased sewer and one-off WWTP construction

The specific CAS systems selected for this study is the Dutch treatment principle called PhoSim with mechanical sludge thickening (STOWA 2007). In this process the influent enters via an inlet structure and undergoes pre-screening and sand trap treatment (Figure 3). In the first anaerobic tank, biologic phosphate removal takes place. The volume of the tank is based on the average dry weather discharge and the required anaerobic residence time (0.5 hours). In the subsequent aeration ditch ammonium is converted to nitrogen gas via nitrification and denitrification. The ditch has an aerated zone, to facilitate the nitrification, so free and saline ammonia is oxidised to nitrite and nitrate;as well as an anoxic zone where nitrate is converted to nitrogen gas via denitrification. The volume of the aeration ditch is based on the desired sludge age (14 days), sludge production and sludge content (4 g/l). In the sedimentation tank the treated water is separated from the sludge. A proportion of the sludge is returned to maintain biological activity. Excess sludge is mechanically thickened before storage. See supplementary material, section 3 for detailed dimensions.

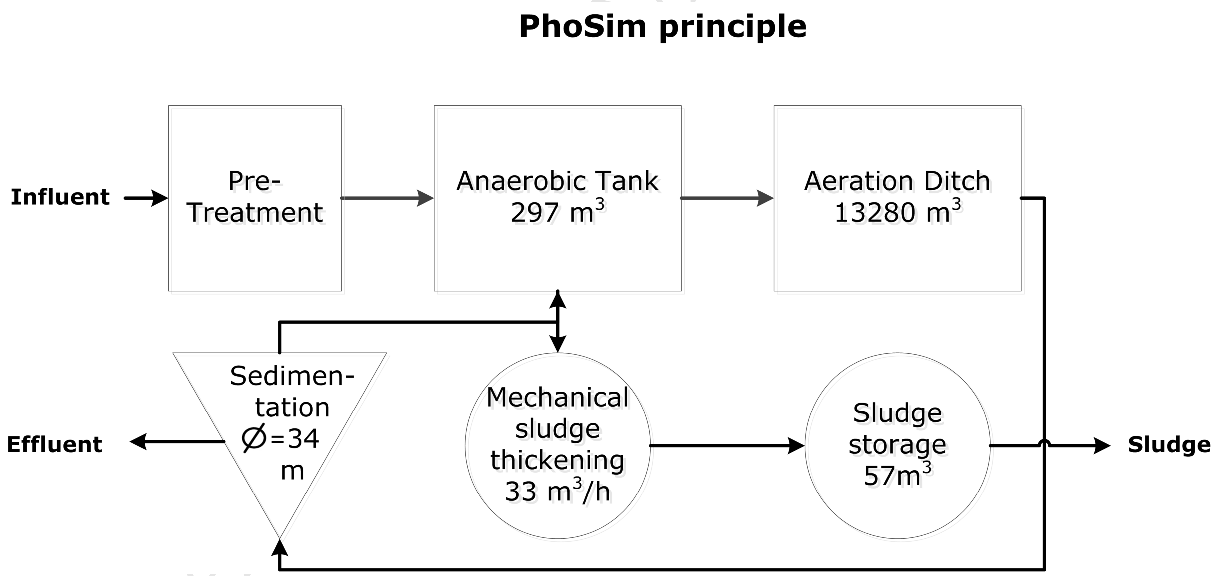


Figure 3: Flow diagram for the CAS wastewater treatment plant, with dimensions as used in this study for a 57548 PE installation.

2.3.2 Source-separated collection and treatment (II)

Decentralised collection and treatment systems are currently used or constructed at pilot sites including Noorderhoek in Sneek - The Netherlands (Butkovskyi et al. 2015, STOWA 2014), Rijnstraat 8 in the Hague - The Netherlands (Nutrient Platform 2016), Jenfeld Au project in Hamburg - Germany (Hamburg Wasser 2016, Klein and Londong 2013) and the project H+ in Helsingborg - Sweden (Hagman 2016). All of these systems make use of separation of GW and BW at household level.

The treatment system selected is an anaerobic treatment for combined vacuumed black water (BW) and kitchen waste (KW – Table 1) and a bio-flocculation/ aerobic treatment (Absorbtion und Belebung – AB (Böhnke 1977)) system for GW as currently operating in a pilot installation for 1200 PE in the Netherlands (Figure 4). KW is included to increase the biogas yield (Kujawa-Roeleveld et al. 2005). The treatment of black water and kitchen waste makes use of an Upflow Anaerobic Sludge Blanket (UASB) reactor, combined with an oxygen-limited autotrophic nitrification/denitrification (OLAND) process that removes nitrogen (nitritation-anammox process) under low COD concentrations present in UASB effluent (de Graaff et al. 2010, Kuai and Verstraete 1998). Biogas is burned to generate heat energy and electricity. The heat generated is used to warm the BW effluent, the electricity is fed it into the electricity grid. The avoided heating costs and the income generated from electricity are accounted for in the economic model (supplemental material). The volume of the UASB is calculated based on the hydraulic retention time [day] and the flow rate [m^3/day]. The biofilm surface [m^2] and specific surface [m^2/m^3] are used to calculate the required volume for the OLAND reactor. To remove phosphorus, Sruvite (magnesium ammonium phosphate) precipitation is applied by adding magnesium (Doyle and Parsons 2002). The volume of the Struvite reactor is determined by the upward flow [m/h], retention time (RT) [h], Struvite production [kg/PE/year] and the Struvite reactor load [kg/m^2/h]. The treated BW is then further treated in the AB system together with GW. The AB system has two functions. Firstly, it is a polishing step for the treated BW and secondly by adding GW to the BW the effluent is diluted thereby reducing the concentration of pollutants in the effluent. In the A-trap suspended material precipitates by bio flocculation. In the B-trap, the remaining organic matter is oxidised and ammonium converted to nitrate (Hernández Leal et al. 2010, STOWA 2014). AB-trap excess sludge serves as input for the UASB. See supplementary material, section 4 for dimensioning details.

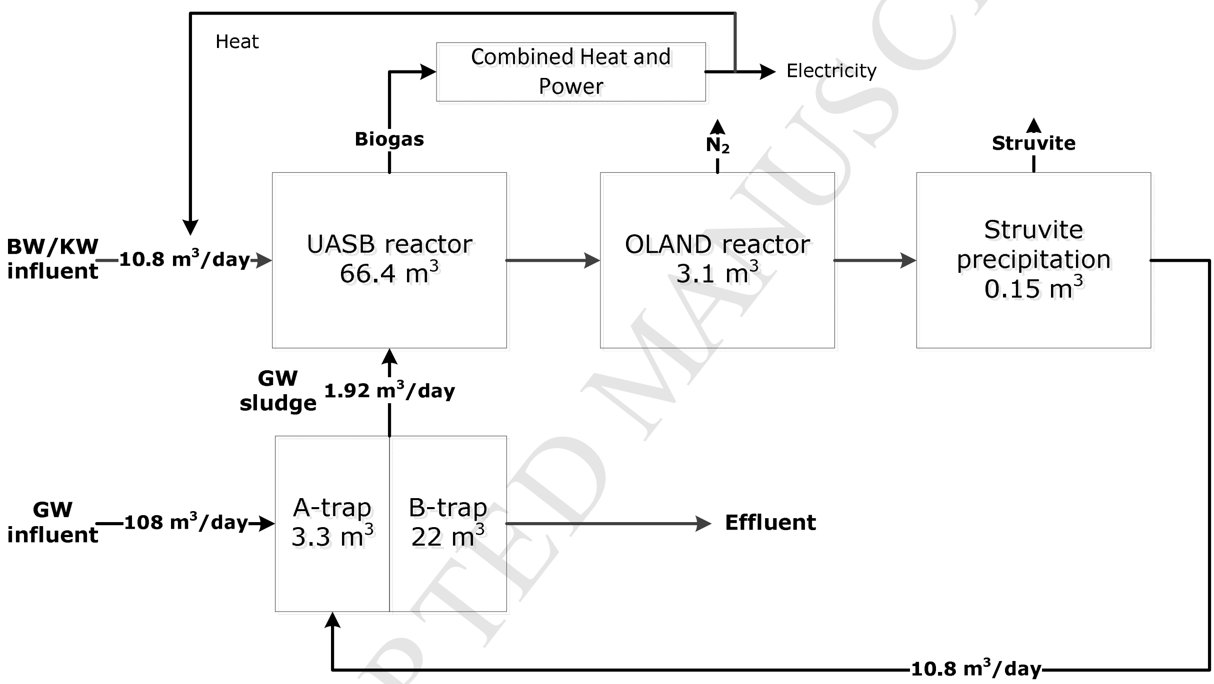


Figure 4 District scale technology scheme for treating source-separated concentrated black water (BW) combined with grinded organic kitchen waste (KW) and grey water (GW) for 1200 PE.

2.3.3 Hybrid system (III)

For this system GW is treated at neighbourhood level of 120 PE. Treatment is performed with a constructed wetland (helophyte filter) of the vertical subsurface flow type (V-SSF). The remaining wastewater (i.e. BW) is transported to the central wastewater treatment plant, where it is treated with the conventional treatment process (section 2.3.1). Constructed wetlands are chosen for GW treatment because their biological mechanisms are perceived to be cheap, ecological, sustainable, innovative, easy to maintain and robust (Nanninga 2011).

Organic removal takes place via the symbiotic relationship between macrophytes and microorganisms living in the root zones. Other mechanisms such as ultra violet radiation, temperatures and physicochemical (anaerobic and aerobic) reactions have an important effect on the purification of the wastewater (Nanninga 2011). The denitrification rate of the V-SSF constructed wetland is poor (Saeed and Sun 2012, Vymazal 2007), but appropriate for GW treatment (GW contains 9% of total nitrogen in wastewater) (Eriksson et al. 2002, Kujawa-Roeleveld and Zeeman 2006). Figure 5 gives schematic the V-SSF installation. See supplementary material, section 5 for dimensioning details.

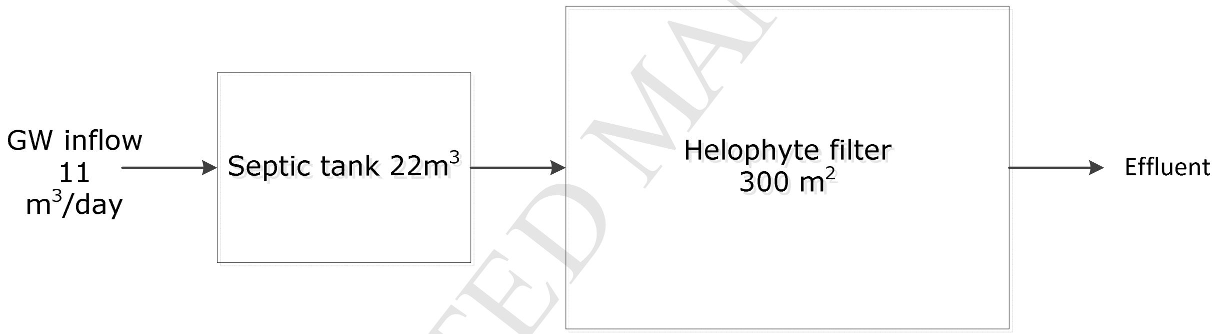


Figure 5 Neighbourhood scale GW treatment with vertical subsurface flow constructed wetland.

2.4 Cost assessment

To determine the lifetime costs of the assets, the design parameters described above are used to determine the capital expenditure (CAPEX) and operating expenses (OPEX). Financial gains included for the cost estimation such as governmental taxes and economic values of recovered nutrients are not included. The impact of cost reduction and revenues is however evaluated in section 3.3.

2.4.1 Sewer system

All sewer systems are separated sewers, thus no rainwater is transported in the systems. Costs for rain water discharge are not included. All costs for sewerage are based on empirical cost models of the umbrella organisation of the Dutch sewerage industry and public bodies (RIONED 2007). Details of the cost calculation can be accessed in the supplementary material, section 2. Sewer costs are estimated for laying new pipes into undeveloped ground. The estimation does therefore not include removal of old pipes. For gravity sewers the pipe material is concrete with an installation of a manhole cover every 40m. Vacuum sewer costs are derived from replacement costs, which are approximately 25% -35% lower, therefore cost are multiplied by 0.7 (RIONED 2007). Costs for construction of the pumping station comprise mechanical and electrical works and costs for constructing the building (RIONED 2007). The operational costs of the sewerage systems are incurred by pumping and cleaning of the pipes. The OPEX for pumping stations comprises the maintenance of the pumping station and costs for electricity for pumping the wastewater. Maintenance of sewer pipes is mainly related to cleaning, which is required before inspection is possible. Sewer inspection frequencies and therefore sewer cleansing frequencies are in the order of 1/10 years for new sewers (Ten Veldhuis 2010).

2.4.2 Treatment

Treatment (CAPEX) and (OPEX) estimations are derived from empirical cost functions provided by Royal Haskoning DHV for the CAS and DESAH[3] (Meulman unpublished) for the decentralised source-separated system. Information for V-SSF constructed wetland is based on information from literature (Nanninga 2011, Rousseau et al. 2004). The empirical cost functions calculate CAPEX of the CAS as follows:

[3] DESAH is specialized in development of decentralized wastewater treatment systems. They altered the empirical cost functions provided by Royal Haskoning DHV such that costs for the decentralised source separated system can be calculated.

$$Capex = a \cdot X^b \qquad \text{Equation 9}$$

Where: X = treatment technology specific parameter determining the investment. The scaling factors ‘a’ and ‘b’ are empirically determined and reflect the economies of scale. Additional costs are made for installing electrical works (40% of CAPEX) and finishing of the treatment works (20% of CAPEX plus electrical works costs). Furthermore, 70% surcharge costs are added over the total aforementioned costs. OPEX costs include: maintenance (0.5% of total civil engineering costs plus 1.5% of total mechanical engineering costs), operating energy (0.08 €/kWh), chemicals (1.5 €/kg Fe and 13 €/kg PE), staff (1full time employment = 45000 €/year), laboratory costs (1200 €/day), dewatering sludge (250 €/ton dry matter) and final sludge processing (400 €/ton dry matter).

CAPEX for the neighbourhood scale V-SSF helophyte filter is 133.4 €/m^2, based on the average cost information from the region Flanders, Belgium (Rousseau et al. 2004). OPEX for the neighbourhood scale V-SSF helophyte filer is 9.1 €/m^2/year, based on two Dutch V-SSF constructed wetlands, located at Eva Lanxmeer Culemborg and Polderdrift Arnhem (Nanninga 2011). CAPEX and OPEX for the septic tank, that is required as a pre-treatment to the constructed wetland are based on Moreira Neto et al. (2012) and van Dijk (personal communication). More detailed costs for the constructed wetland can be found in the supplementary material.

2.4.3 Discounted lifetime costs

To determine the lifetime costs, the CAPEX and OPEX for all collection and treatment systems are discounted using Equation 10.

$$DLC = \sum_{t=1}^{T_p} \frac{Expenses_t}{(1+r)^t} \qquad \text{Equation 10}$$

Where: DLC = discounted lifetime costs, r = the discount factor of 5% and T_p = planning horizon (30 years). Revenues are not included since the water sector is a natural monopoly where the revenues are usually determined by the costs (Maurer 2009).

3 Results and discussion

3.1 Discounted lifetime costs - deterministic exponential city growth

3.1.1 Sewer network

Results show that at a growth rate of 2% the conventional and source-separated system have similar discounted lifetime costs, while the hybrid system is clearly more costly (Table 3). The total discounted lifetime cost for the source-separated system is 2% higher than those of the conventional system, while the hybrid system is 70% more expensive. In all cases sewer related costs make up the biggest share, on average 63% of the total costs, with the conventional single pipe network accounting for 59% of the cost, the source-separated system for 69% and the hybrid systems for 62% (Table 3). More than 80% of the network costs are incurred by the sewers at the neighbourhood level, due to its high overall length (657m per neighbourhood or about 330km for the entire expansion area). Conversely, expenses for connecting between districts and to the WWTP in the hybrid and conventional system account for only 10% and 4% of the sewer costs (or 6% and 2% of total costs). The implication of this cost distribution for network infrastructure is that the benefits of phasing the sewer system in the source-separated and hybrid system are limited. In the source-separated system, inter-district pipe and an artery pipe are not required (see Figure 2 – for all sanitation systems growth of the required sewer network takes place in phases). But as these pipes make up only a small part of the total investment, the benefits from phasing cannot offset the higher expenses for the required double piping at the neighbourhood and district scale where the major share of the costs is incurred. In the hybrid system the profit of phasing is negligible as it requires pipes at all scales, with only the inter-district and artery having a smaller diameter (see supplemental material section 2). The benefits of phasing are therefore mainly found in the treatment infrastructure.

3.1.2 Treatment assets

Expenses for treatment are lowest for the source-separated system (ca. €27 million), while the conventional system is 28% more costly (ca. €34million). The hybrid system is 101% (ca. €54million) more costly and therefore clearly not competitive with the other systems. When considering the wastewater treatment only, it can be concluded that the lower discounted lifetime costs for the source-separated system are a result of phasing the implementation in smaller increments. In other words, they are a result of delayed asset investment (CAPEX) and a resulting lower OPEX. The CAPEX benefits are evident in the fact that the net CAPEX, or the undiscounted CAPEX, and the discounted CAPEX of conventional treatment are both ca. €12 million (Table 3– all investments take place in year 0 for the conventional treatment). Contrary to this the discounted CAPEX for the source-separated system is ca. €16 million, while the net CAPEX is ca. €33 million or 168% that of the conventional treatment. The development of discounted costs over time offers an explanation for the difference between total net expenses and total discounted expenses (Figure 6). The one-off investment for central WWTP at the start of the development for the conventional and hybrid system is a cost item that does not benefit from discounting. After year one, there is a sharp decline in costs followed by a more linear slow decline. This last part of the curve is only the operational costs that benefit from discounting.

The source-separated treatment shows a different pattern. Discounted expenses are more constant over the lifetime, because there is no large investment at the start and annual discounted costs vary between €0.68 million and €1.29 million over the entire period. This variation is a representation of the delayed investment and phasing that results in new districts being built each year (between t_x - t_{x-1}; Figure 6). Due to the exponential population development there are more people per time intervalmoving into the area at the end of the planning horizon than at the start. Therefore, in the years 20-30 two new neighbourhoods are constructed annually at approximately the same costs as a single neighbourhood at the start of the planning horizon. In other words, at the end of the planning horizon the cost for operation of all districts and the construction of new district treatment are the same as at the start of the planning horizon. Figure 6 further shows that the cost advantage of the source-separated treatment lies in the first half of the asset lifetime. It takes until year 14 that the annual operating and investment costs of source-separated treatment are continuously above that of the centralised conventional system. The reason for the higher costs of the conventional system is that during the first half of the planning horizon the absolute number of PE served is small when compared to later on in the planning period. As a result, much idle capacity must be maintained leading to a high OPEX of the conventional system. Indeed, this OPEX is higher than the combined CAPEX for new source-separated systems and the OPEX of existing source-separated systems until year 14.

Table 3: Discounted lifetime costs at the end of planning horizon with exponential growth 2%. The table provides a breakdown of cost in the sewer system and the relative cost share for the infrastructure (conventional = centralised treatment of municipal sewage, source-separated = district scale treatment of GW and BW, Hybrid = Neighbourhood treatment of GW and centralised treatment of BW, see also Figure 1 and section 2.3 for more details).

			Conventional			**Source separated**			**Hybrid**		
Expenses	*Scale*		CAPEX	OPEX	Share	CAPEX	OPEX	Share	CAPEX	OPEX	Share
Sewer	*Neighbourhood*	Neighbourhood connection and public sewer	€ 22,289,000	€ 226,000	45%	€ 27,248,000	€ 1,926,000	49%	€ 44,404,000	€ 432,000	50%
		Private sewer	€ 18,826,000	€ 210,000	38%	€ 23,940,000	€ 1,957,000	44%	€ 37,653,000	€ 402,000	43%
	District	District backbone	€ 1,793,000	€ 18,000	4%	€ 2,189,000	€ 155,000	4%	€ 1,790,000	€ 17,000	2%
		pumping station	€ 1,331,000	€ 392,000	3%	€ 1,353,000	€ 298,000	3%	€ 773,000	€ 376,000	1%
	City	Inter district pipe	€ 2,311,000	€ 23,000	5%	€ -	€ -	0%	€ 1,387,000	€ 111,000	2%
		District connection	€ 25,000	€ 5,000	0%	€ -	€ -	0%	€ 25,000	€ 5,000	0%
		Area backbone	€ 2,382,000	€ 123,000	5%	€ -	€ -	0%	€ 1,678,000	€ 118,000	2%
	Sub-Total sewers		€ 48,957,000	€ 997,000	**59%**	€ 54,730,000	€ 4,336,000	**69%**	€ 87,710,000	€ 1,461,000	**62%**
	Total		**€ 49,953,000**			**€ 59,065,000**			**€ 89,170,000**		
Treatment			€ 12,240,000	€ 22,216,000	41%	€ 16,131,000	€ 10,754,000	31%	€ 22,283,589	€ 31,799,663	38%
	Total		**€ 34,456,000**			**€ 26,885,000**			**€ 54,083,000**		
Total overall			**€ 84,410,000**		**100%**	**€ 85,950,000**		**100%**	**€ 143,254,000**		**100%**

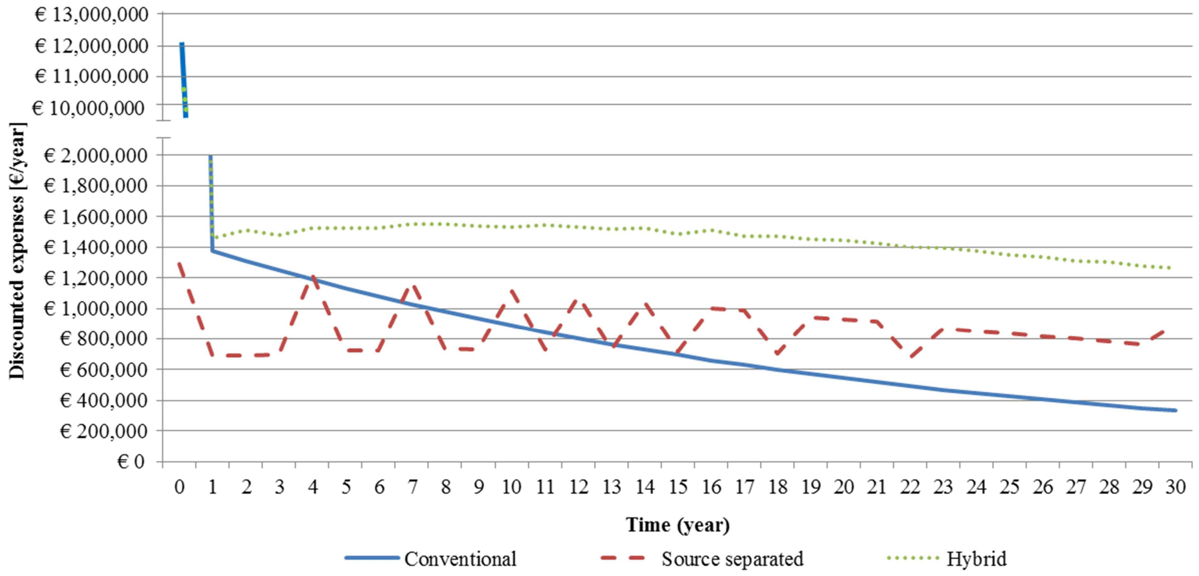


Figure 6 Discounted treatment expenses over the planning horizon with exponential growth of 2%.

3.2 Discounted lifetime costs – variable urban growth

Figure 7 shows the same results as Figure 6, but with a variation around a 2% growth rate and 1000 runs of a Monte Carlo Simulation. The resulting S-curve is a cumulative distribution function of these runs. It shows at which probability a value equal to or lower than a certain discounted cost occurs. For example, it shows that there is a 50% chance that a conventional collection and treatment system will cost about €100 million or less. The most cost-effective option is the conventional system (Figure 7) followed by the source-separated system and the hybrid system. A noteworthy observation of Figure 7 is that discounted expenses are higher compared to the results with standard exponential growth (Table 3). The average discounted expenses are ca. € 100 million for the conventional system, ca. € 114 million for the source-separated system and ca. € 182 million for the hybrid system. The difference between the conventional and the source-separated systems increased from 2% to 12% and the difference between the conventional and the hybrid systems increased from 70% to 79%. The reason for this change is that, because growth does not follow a perfect exponential curve, but rather deviates from it, investments in phased design are made earlier (Figure 8). These earlier expenses result in lower benefits from discounting. This effect is visualized in Figure 8, showing two variable development scenarios compared to the exponential growth (Equation 1). The conventional system does not respond to variable growth because of one-off construction of treatment. Contrary to this, for the source-separated and hybrid systems new construction of treatment must be made earlier than in a simple exponential growth model, therefore increasing the difference in costs between the systems.

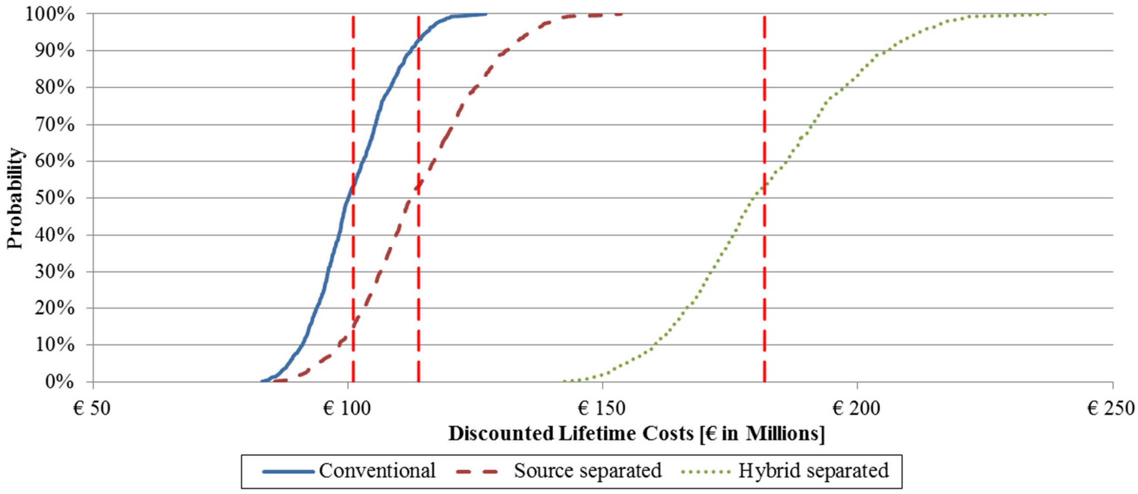


Figure 7 Cumuative distribution functions of 1000 Monte Carlo simulation runs of discounted lifetime cost for each sanitation systems with variable growth around 2.0%.

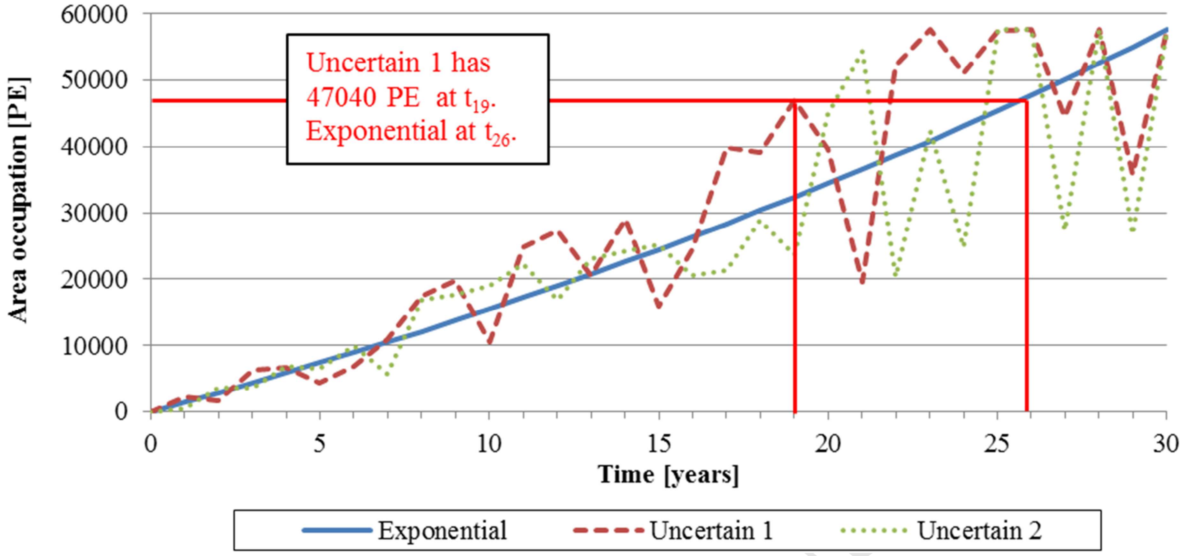


Figure 8 Area development scenarios: Exponential growth (2%) and two simulations of random growth around the exponential growth line. The solid horizontal line provides an example of how uncertainty can reduce the advantage of discounting, because the same PE need to be provided 7 years earlier.

The analysis above assumes that the predicted growth, with some variation over time, actually takes place and that by the end of the asset's life time a plant utilisation of 100% has been reached (Equation 1, Equation 2). This model represents the ideal case of a prediction being met, however, evidence from literature shows that this is often not the case (de Neufville and Scholtes 2011, Panebianco and Pahl-Wostl 2006). Figure 9 shows the situation if these predictions are not met. It can be seen that for growth rates of 0.5% and 1.0% source-separated systems outperform the conventional system. When the growth rate is increased to 1.5%, the likelihood that the source-separated system outperforms the conventional system is 73% or less, at a cost of ca. € 82 million. If the growth rates are further increased the margin between the source-separated and conventional system further increases in favour of the conventional system. This suggests that if growth rates are lower than expected the phasing of the source-separated system creates sufficient cost advantage to outcompete the conventional system. In other words, in situations where uncertainty is large and where there is a risk that the population declines, the source-separated system is more likely to be cost-effective compared to the not phased conventional alternative (Maurer 2009).

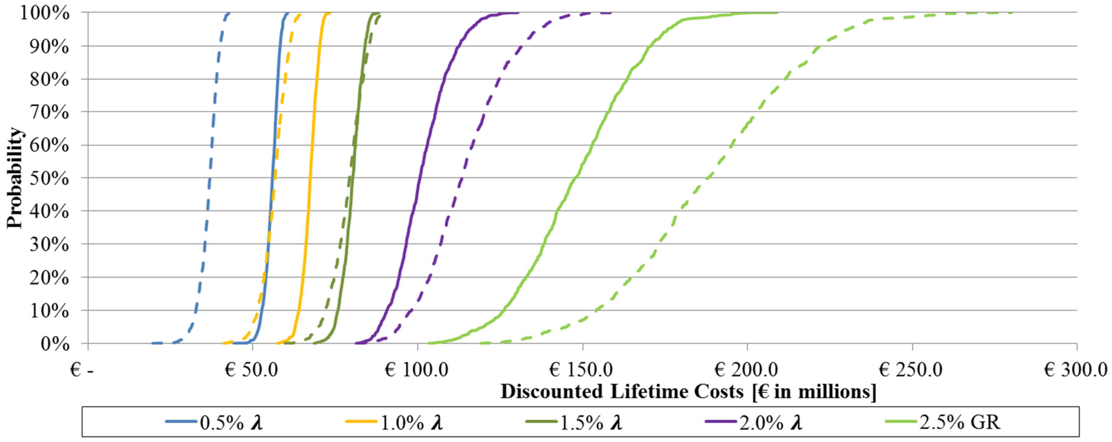


Figure 9 Cumulative distribution functions of discounted lifetime costs for conventional [−−−] and source-separated [- - -] collection and treatment systems results, when prediction for 2% growth are not met. For this analysis the hybrid system was excluded, because it is much higher costs.

3.3 Potential cost reduction

The results showed that DLC are lowest for conventional systems. This analysis did not take into account the potential costs reductions. For example, in the source-separated system costs on curb side organic waste collection may be avoided as kitchen waste is gathered with the vacuum sewer systems and digested together with the black water. De Graaf and van Hell (2014) estimate the cost savings from this to be about 10%. Small cost reductions can further be achieved (<1%) as a result of a result of lower vacuum toilet flushing demand (i.e. ≈1l/flush instead of 6l/flush), leading to savings in operational cost of drinking water production. Furthermore, it is likely, that as source-separation pilot projects are implemented more widely, cost for materials, infrastructure and construction can be further decreased (de Graaf and van Hell 2014). Figure 10 shows the required cost reductions for the source-separated system to become competitive with the conventional system. It can be seen that an overall cost reduction of 12% gives the source-separated system a probability of 63% to be more cost effective than the conventional system. This implies that the source-separated system has the potential to break even with the conventional system, when the evaluation does account for further cost reductions (e.g. incl. drinking water and solid organic waste collection). A choice in favour of the sources separated system is even more likely when the evaluation is expanded to include the sale of products recovered from wastewater, including the sale of struvite as a fertilizer or potentially also the digested sludge. However, currently these products do not have an economic value. The Struvite produced at the source-separation pilot plant in the Netherlands cannot be sold at present. At the current market value of €50/t, suggested for the case of Belgium, the income would lead to a cost reduction of < 0.1% (Geerts et al. 2015). When a the value of Struvite is estimated from the fertiliser prices (about €290 per ton at €1.60 per kg phosphorus and €1.48 per kg nitrogen (Verstraete and Vlaeminck 2011)), then the income generated from Struvite production would account for less than 0.5% of the total discounted lifetime costs.

Another cost reduction that affects the results and thereby the choice of wastewater collection and treatment systems is the sewer network, because it accounts for about 60% of the total expenses. In this research standard pipe diameters are used for gravity sewerage (e.g. 110mm and 200mm), as this is current practice. However, sewer systems could be designed more closely to the actual wastewater flows, based on models for drinking water end use (Blokker et al. 2009). This may result in smaller diameter pipes, thereby reducing costs.

In addition to cost evaluations environmental aspects should be accounted for when selecting a wastewater collection and treatment systems. In the current analysis the lower effluent pollution loads as well as the better sludge quality of the source-separated system are points in support for selection of this system. In combination with cost reductions and sales of product this may make source-separated systems viable also at high urban growth rates. Furthermore, a Life Cycle Assessments of the wastewater collection and treatment can show the impact of materials, construction and operation on a broad suit of environmental endpoints. Using such an analysis offers an important additional perspective on wastewater collection and treatment system performance than simply costs.

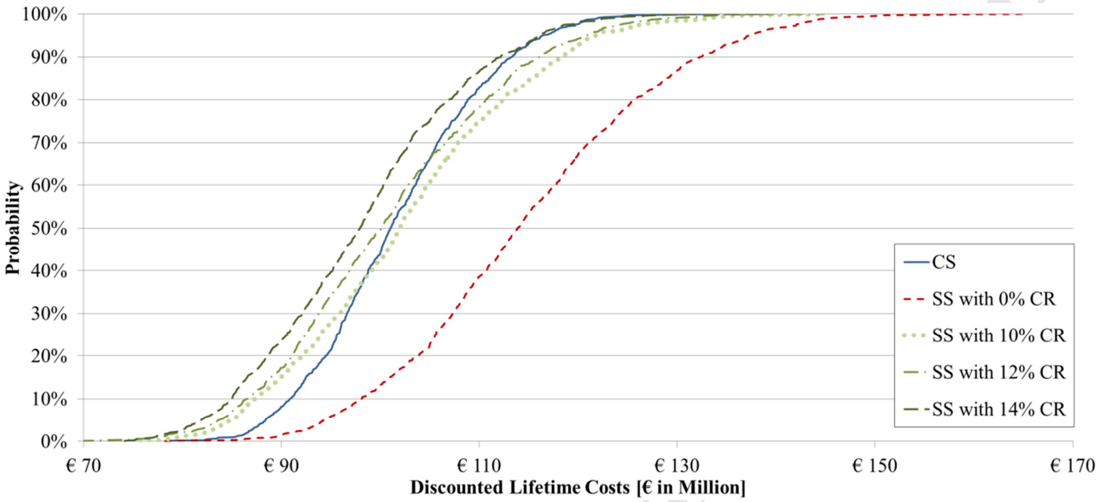


Figure 10 Cumulative distribution functions of 1000 runs of discounted lifetime costs at Growth Rates of 2.0% for the Conventional System (CS) and Source-separated System (SS) with Cost Reductions (CR).

4 Conclusion

- Conventional systems have the lowest discounted lifetime costs when predicted population growth becomes reality or is exceeded.
- If population growth is 0.5% lower than expected then source-separated systems are more cost effective than conventional systems, because of reduced idle capacity.
- Variable urban growth patterns (uncertainty) increase discounted lifetime costs as it requires earlier investment, therefore reducing benefits from discounting in favour of the conventional system.

- Phasing advantages are mainly in the treatment system, due to the small cost share of sewers that connect between districts and to the centralised WWTP.
- Hybrid systems are not competitive due to double investments in treatment and collection system.
- If cost reductions for source-separated systems are realised and environmental benefits are accounted for, then they can be a viable alternative to conventional centralised sanitation systems also at higher urban growth rates.
- Future research should explore the impact of cost reduction in more detail.

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Highlights

- Conventional system has the lowest discounted costs when growth is as expected
- Source-separated system is cost effective when growth is 0.5% lower than expected
- Variable urban growth patterns (uncertainty) increase discounted lifetime costs
- Cost advantages of phasing infrastructure role out are mainly in the treatment system
- The hybrid system is not competitive