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FACULTY OF CIVIL ENGINEERING

DEPARTMENT OF SANITARY AND ECOLOGICAL ENGINEERING

ENVIRONMENTAL, ENERGY AND OPERATION RELATED ASPECTS OF DECENTRALIZED MANAGEMENT OF WASTEWATERS AND STORMWATERS IN SMALL MUNICIPALITIES

DIPLOMA THESIS

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Práce bude obsahovat teoretickou a praktickou část. Teoretická část bude zaměřena na rešerši možných decentrálních způsobů hospodaření s odpadní a dešťovou vodou v městském povodí. Praktická část se bude sestávat z popisu metodického postupu a z výsledkové části.
Cílem této diplomové práce je posouzení environmentálních, energetických a provozních aspektů odvodnění malých obcí při různém stupni centralizace, a to na příkladu vybraných lokalit.

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ABSTRACT

This thesis focuses on environmental, energy and operation-related aspects of decentralised wastewater and stormwater management in small municipalities. The aim of this thesis is to compare centralised and decentralised wastewater and stormwater management in municipalities with less than 500 PE.

First part of the thesis explains the rationale for decentralisation of wastewater and stormwater management, describes traditional centralised approach to wastewater and stormwater management, presents basic concepts and technologies used for decentralised wastewater management, introduces sustainable urban drainage systems, summarises Czech and European legislation related to wastewater and stormwater management, and describes the authorisation process for wastewater treatment plant construction and discharges of treated wastewater.

Second part compares centralised and decentralised wastewater and stormwater management based on environmental, energetic and operational reliability criteria. Three cases studies are used to calculate pollution loads to receiving environment, energy requirements, and system reliability of centralised and decentralised option.

KEY WORDS: decentralized wastewater management, SUDS, WWTP reliability, energy consumption of wastewater treatment plant

ANOTACE

Diplomová práce se zabývá environmentálními, energetickými a provozními aspekty decentralizovaného nakládání s odpadními vodami v malých obcích. Cílem práce je porovnat centralizované a decentralizované způsoby nakládání s odpadními a dešťovými vodami v obcích s méně než 500 p.e.

První část práce jmenuje výhody a nevýhody decentralizovaného čištění odpadních vod a nakládání s dešťovými vodami, popisuje tradiční přístup k městskému odvodnění a koncepty a technologie používané pro decentralizovaný způsob nakládání s odpadními a dešťovými vodami. Součástí práce je přehled české a evropské legislativy vztahující se k problematice městského odvodnění a popis procesu povolování výstavby ČOV a vypouštění odpadních vod v České republice.

V druhé části práce je na třech konkrétních případech provedeno porovnání a zhodnocení centralizovaného a decentralizovaného způsobu nakládání s odpadními vodami z hlediska produkovaného znečištění, energetických nároků a spolehlivosti jednotlivých systémů.

KLÍČOVÁ SLOVA: decentralizované nakládání s odpadními vodami, SUDS, spolehlivost ČOV, energetické nároky na provoz ČOV

Contents:

1 INTRODUCTION

The accession of Czech Republic (CZ) to the European Union (EU) in 2004 had a major influence on development of public infrastructure for collecting and treatment of municipal wastewaters. To comply with EU legislation, it was necessary to accelerate the implementation of sanitation infrastructure projects. The percentage of population connected to a sewage network rose from less than 78% in 2003 to 84.7% in 2016 [17]. However, year-on-year increase in the share of population with access to public sanitation network has been stagnating in the past years [12], since most of the major settlements are now in compliance with the obligation imposed by the EU legislation. Recently, the focus has been turning to small municipalities where the implementation of wastewater collecting and treatment systems is technically and economically more challenging.

The Urban Waste Water Treatment Directive 91/271/EEC (UWWTD) set staged deadlines for implementation of wastewater collecting systems, secondary treatment, and more stringent treatment facilities in sensitive areas. Agglomerations of more than 10,000 PE discharging into sensitive areas were obliged to comply with the Directive by the end of 1998, agglomerations of more than 15 000 PE discharging to normal areas were due to compliance by the end of 2000 and agglomerations with PE above 2000 were due to compliance by December 2005. [93] However, the Directive has not set any binding deadlines for agglomerations of less than 2,000 PE.

For Czech Republic and 12 other member states, transition periods were negotiated mainly due to the significant financial burden imposed by implementation of sanitation infrastructure. The transition period set for Czech Republic ended in 2010 and as of now, wastewater treatment in agglomerations of more than 2000 PE has been mostly resolved. According to the 8th Technical Assessment of Information on the Implementation of Council Directive 91/271/EEC, all Czech agglomerations of more than 2000 PE had collecting systems in 2015. For the obligation of providing secondary or equivalent treatment to discharges from agglomerations of more than 2000 PE, the compliance rate was 87%. However, CZ showed poor compliance (54%) with the requirement of more stringent treatment for wastewater discharged into sensitive areas. [93]

The UWWTD obliges the EU member states to ensure appropriate treatment of wastewaters produced in agglomerations of less than 2000 PE before they can be discharged to freshwater bodies. Moreover, it states that treated wastewater shall be reused whenever appropriate. However, no obligatory deadline for ensuring appropriate treatment of wastewater has been set.

The motivation for public investment in wastewater structure is therefore not motivated by financial penalization in case of failure to comply with this Directive. Rather than that, is it driven by the Water Framework Directive 2000/60/ES requiring the member states to undertake precautions to prevent deterioration of the environmental status of waterbodies or to maintain their good status.

Currently, majority of all the Czech municipalities of less than 2000 PE have insufficient or no proper wastewater treatment infrastructure. There were 5562 municipalities of less than 2000 PE in total in Czech Republic in 2016 [13], of which more than 3000 municipalities were not equipped with sewage network and/or WWTP [30]. The untreated wastewater is often discharged either directly or indirectly through cesspits and septic tanks into the receiving waterbodies.

The implementation of wastewater collecting and treatment systems in small municipalities is a challenge mainly from the financial and institutional point of view. Small municipalities often have very limited budgets that do not allow them to build, operate and maintain infrastructure and facilities needed to meet the increasing environmental protection demands.

According to The State Environmental Policy of the Czech Republic 2012 – 2020, projects dealing with centralised wastewater treatment in agglomerations with less than 2000 EO will be supported only if it is duly justified technically and economically in relation to an alternative to individual wastewater disposal systems, especially in areas requiring special protection. [56]

Small municipalities are often characterised by low population density, meaning the producers of pollution have greater distances between each other, and to construct a sewage network connected to a central WWTP would impose large construction costs per PE, while only connecting a small portion of population [35], [46], [6]. That is why recently the suitability of these systems for rural areas has been brought into question.

Recently, the state-of-art centralised approaches to urban water management have been questioned also in terms of sustainable development, due to their environmental impacts, significant costliness of wastewater structure implementation, necessity of long-term planning and possible vulnerability. The concept of decentralised urban water management has been proposed and promoted by many water management experts and scholars [62], [90], [11], [49].

On the other hand, traditional approaches have been fostered for decades for their centralised management, economies of scale and high standard of effluent quality. Moreover, large sewerage and WWTP schemes are usually widely supported by legislative measures, while decentralised

wastewater treatment usually has small or zero administrative support, which may discourage citizens and decision makers [35].

The aim of this thesis is to compare centralised and decentralised wastewater and stormwater management in three selected Czech municipalities based on environmental, energy, and operation-related aspects.

2 THEORETICAL BACKGROUND

2.1 Influence of urbanization on natural hydrological cycle

In natural conditions, a certain part of rainfall is allowed to infiltrate the soil, another part evaporates back into the atmosphere during the evapotranspiration process. After all the depressions fill up with water and start to overflow, surface runoff occurs. In urbanized areas, this natural cycle is severely altered. Unlike in rural areas, there is not enough vegetation to support sufficient transpiration. Most surfaces do not allow infiltration, either because of used impervious materials, or due to high level of compaction. As a result, the possibility of infiltration and evapotranspiration is strongly limited, and majority of rainfall flows out in the form of surface runoff. This results in higher peaks and shorter concentration times of the runoff hydrograph, leading to higher possibility of flooding downstream and more frequent and severe drought periods. Altered evapotranspiration rates cause changes in microclimate due to lower humidity and higher temperatures.

It has been predicted that in the future there will be a higher occurrence of extreme rain events due to global warming. Existing sewage systems will not be able to cope with the future conditions, and without modifications, urban areas would experience frequent urban flooding due to insufficient capacity of the system. In the current situation, where majority of rainfall is transported from urban areas in the form of surface runoff, and evaporation and infiltration are very limited, this would become a very serious problem [108]. Expanding the capacity of existing centralised urban water systems would be, in many cases, very expensive and labour-intensive, and it would not solve the problem but merely transfer it to a downstream area.

Another environmental issue caused by rapid transportation of both stormwater and wastewater from urbanised areas by combined sewage networks is overflowing of combined sewer overflows (CSOs). When the designed hydraulic capacity of a combined sewer is reached, the untreated wastewater water diluted by stormwater is discharged through a CSOs into the nearest waterbody. Aquatic organisms living down the stream can suffer from hydraulic stress and the overflowing polluted water can cause acute water quality impacts.

Furthermore, urban areas are characterised by high population density. Accumulation of people results in production of large amount of wastewater that needs to be managed to prevent spreading of diseases and odour nuisances. Wastewater contains high amounts of pathogens from

human waste, and as such represents a hygienic threat and must be treated prior to discharge. Domestic wastewater also contains high concentrations of nutrients that need to be removed to limit the environmental impact of human activities on the environment. In the past it was recognized that nutrients entering waterbodies due to various anthropogenic activities cause eutrophication and destabilise natural aquatic ecosystems. [20] Rain events in urban areas with combined sewage networks represent flooding and hygienic risk, because when the network is hydraulically overloaded, mix of stormwater and wastewater flows up to the surface.

Without urban water management, the stormwater and wastewater would accumulate in cities. Traditionally, the approach to this problem was to convey both wastewater and stormwater away from the city as quick as possible, using gullies and systems of underground pipes.

2.2 Centralised systems

First drainage networks were implemented already in ancient Rome. The Romans understood the importance of hygiene for prevention of epidemics very well - Rome was drained of its wastewater and stormwater using a vast sanitation network, and even the smaller cities in vicinity of Rome had at least public latrines. Unfortunately, after the collapse of Roman empire, all the sanitation knowledge waned, and disused and unmaintained aqueducts and sanitation systems began to deteriorate. [48], [53]

Until the end of 19th century there were almost no developments in the field of urban drainage, and water itself was deemed to be “unhygienic”, which led to many epidemic outbreaks [48]. Most households did not have any type of sewer or latrine. Instead, they used chamber pots, which were emptied on the street, together with other household waste. During a storm, the waste was swept into the nearest waterbody.

After the industrial revolution in the 18th century, the situation got uncontrollable. Following the two leading epidemics of cholera in major European cities in 1832 and 1849, construction of first centralised drainage systems in European capitals began in the latter half of 19th century [69].

From historical point of view, it was desirable to rapidly convey stormwater and wastewater away from the city. This approach ensured that possible pathogens will be transported downstream to eliminate the threats to public health, and that urban flooding will be prevented. Extensive drainage networks were designed to transport the urban water, which led to centralization of urban water treatment. [5]

In a centralised system, wastewater is collected in a **wastewater collection system** and transported to a central facility. The choice of collection system depends on the terrain setting, geological conditions, and position of groundwater table.

Conventional gravity collection system does not require energy input, but usually has high installation costs because of deep excavations [84]. Gravity systems must have a minimum slope to maintain minimum velocities in the sewer to avoid settling down of particles which would result in blockages. In areas with high groundwater table, the use of conventional gravity sewers is limited, as they are prone to infiltration/inflow (I/I).

Where the flat or undulating terrain does not support gravitational flow or where it would not be economical to excavate deep and wide trenches needed for conventional gravity sewer construction, e.g. in rocky terrain or densely populated urban areas, pressure or vacuum sewers must be used.

Pressure systems require pumping to transport the wastewater to WWTP. Usually, each household has a *pressure system unit*. Wastewater flows by gravity to the pressure system unit with a grinder pump where it is stored until the unit is full. Before being pumped into the pressurised system, the wastewater is grinded to prevent clogging of small-diameter pipes. An alternative version of this arrangement is a septic tank effluent pumping (STEP) system [91]. Wastewater is treated on-site in a septic tank that discharges into an underground tank, from where it can be pumped straight into the pressure network without grinding, as most of the solids are removed in the septic tank.

In vacuum collection systems, wastewater is drawn from the collection points toward the WWTP under vacuum. Wastewater from each household is collected in a sump. When wastewater reaches a specific level in the sump, the valve opens and the liquid is sucked towards a central vacuum lift station. [32] From the central vacuum lift station there is a pressure sewer to transport the wastewater to WWTP.

CENTRALISED TREATMENT TECHNOLOGY OVERVIEW

Variety of treatment technologies is available and the choice of most suitable technology depends on wastewater characteristics, effluent quality requirements, available land and possible future tightening of effluent requirements. [71] *Figure 1* shows an example configuration of treatment processes for centralised WWTP.

In the **preliminary treatment** step, wastewater flows through screens which remove large solid pieces of waste, such as rags, plastic bottles, cans, branches,,

sanitary towels and tampons, and animal carcasses. Then, it flows through a grit chamber, where the flow is slowed down or redirected to allow grit and sand to settle down. This protects the mechanical equipment in further treatment stages, and also eliminates amount of anorganic particles in primary sludge. Preliminary treatment can also encompass some sort of oil and grease separation, e.g. by dissolved air flotation (DAF).

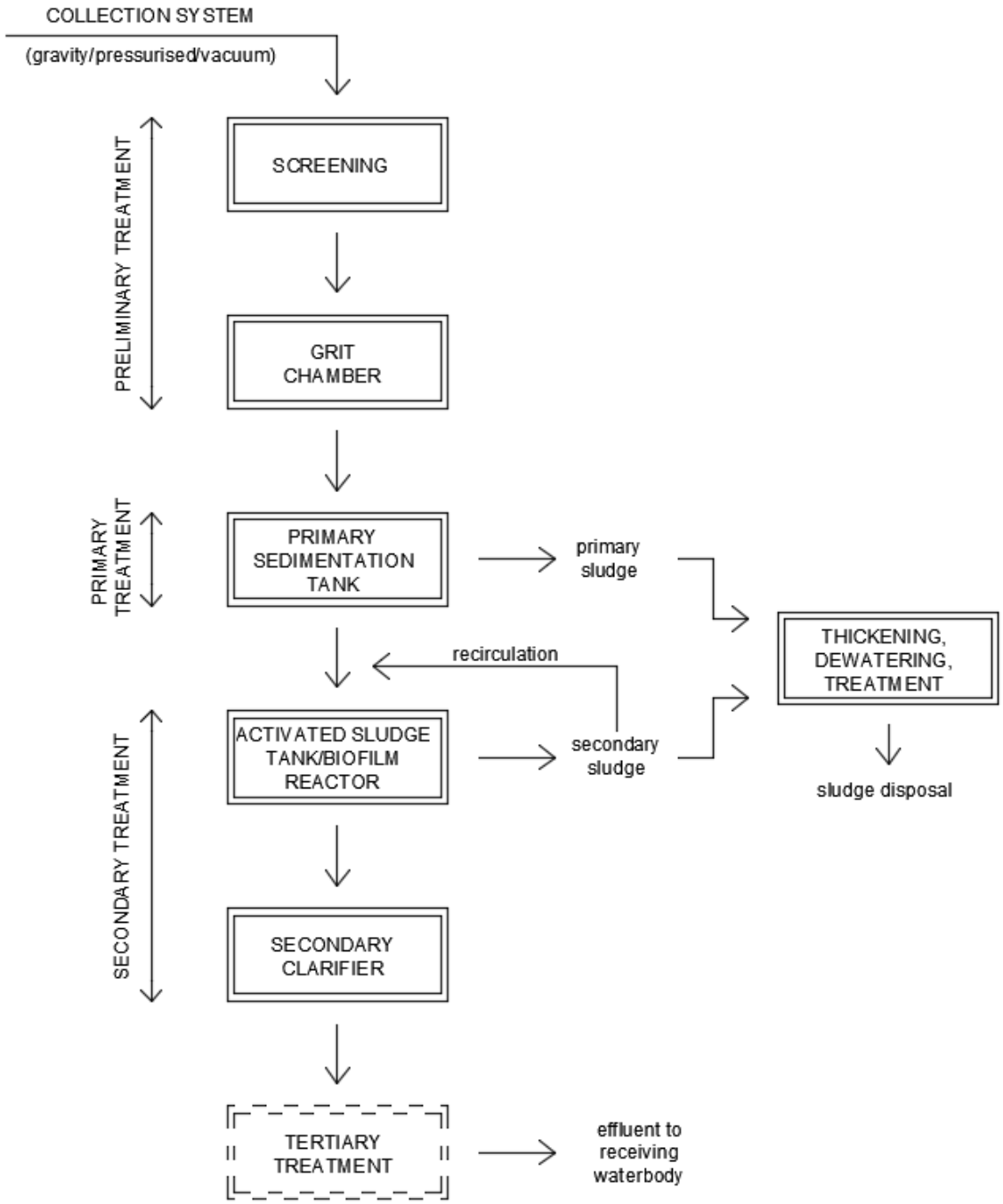

Figure 1. Example schematics of a centralised WWTP

During the **primary treatment** process, part of the suspended solids and organic particles is removed by simple sedimentation. Smaller particles tend to coagulate during settling, and larger aggregates settle down more easily. Primary treatment normally removes up to 95% settleable solids, 40 to 60% total suspended solids, and 25 to 35% BOD_5 [84].

Secondary treatment utilizes natural biological processes by creating favorable living conditions for microorganisms that are capable of degrading biological matter that cannot be removed by sedimentation, such as dissolved substances and unsettleable particles. The dominant technology used for secondary treatment in municipal WWTPs in Czech Republic is activated sludge process [103], other technological options are biofilm reactors, rotating biological contactors (RBC) or natural-like systems, such as constructed wetlands or stabilisation ponds. Secondary clarifiers are used to remove biomass created in biological tanks, and part of the secondary sludge can be recirculated to increase the influent concentration of wastewater.

Depending on required effluent quality, some WWTPs may have some form of **tertiary treatment**, such as membrane filtration, ozone disinfection, chlorination or wetlands and stabilisation ponds as tertiary treatment step.

Sludge treatment and final disposal includes range of methods that are used to stabilise and hygienise sludge and prepare it for reuse or disposal. Prior to stabilisation, sludge is usually thickened by gravity or by mechanical thickeners up to 5-6% dry solids (DS) [34]. Stabilisation of sludge reduces the on-going processes in sludge to a minimum, hygienisation ensures that pathogens are eliminated to a certain harmless level. Commonly used techniques providing both stabilisation and hygienisation are anaerobic and aerobic digestion, composting, and lime stabilisation. Anaerobic sludge stabilisation in digesters reduces the volume of sludge, improves its dewatering properties, and offers the possibility to capture biogas which can be then used to produce heat and electricity in a CHP unit. Aerobic stabilisation provides sludge with worse dewatering properties and requires longer retention times as compared to anaerobic digesters [21]. The aerobic process can take place in activated sludge tank or in a dedicated tank for aerobic stabilisation. Aerobic digestion is usually used in smaller WWTPs.

Sludge is then dewatered to 20 to 50% DS [34], using either natural dewatering in lagoons, or mechanical presses and centrifuges. Chemical flocculants can be added. Another option is thermic drying which can be combined with preceding dewatering centrifuge.

Alternatives for final disposal of sludge are land application, incineration, or landfill. Co-incineration with other fuels can offset the environmental impact by producing some energy, and the sludge

ash is sometimes used in construction materials. Sludge can be also tranformed by thermal drying to produce pellets which can be used in agriculture or as a fuel. As far as landilling is concerned, Directive on the landfill of waste (1999/31/EEC) recommends other methods for disposal of biodegradable waste and prohibits landfilling of both liquid waste and untreated waste. In Czech Republic, landfilling of wastewater sludge is prohibited [57], and only marginal part of produced sludge is incinerated. In the period 2009-2013, only 148 tonnes of sludge were incinerated, whereas almost 40,000 tonnes were used in agriculture [23].

Centralised stormwater management measures are implemented together with drainage network, either in-stream or adjacent to the network. Usually, retention/detention basins are implemented to minimise environment pollution and hydraulic stress.

In separate sewage networks, retention ponds or detention-sedimentation basins can be implemented. Retention basins have a stable pool of water. During a rainfall event, the water accumulates in the pond and then it is gradually released, while the pollutants from surface runoff are allowed to settle down, and are broken down in natural processes in the pond. This reduces the pollutant load to the receiving waterbody and reduces hydraulic stress. However, the pond represents a discontinuity in the natural watercourse which can have adverse impact on aquatic environment.

Detention-sedimentation basins located on separate networks do not have a stable pool of water, and they are designed to gradually discharge all water after the rainfall event has ended. Their purpose is to retain solids and settleable particles, and to lower the peak outflow.

In a combined system, wastewater is mixed together with stormwater. During a heavy rain, the wastewater is very dilute, and after the capacity of the pipe is exceeded, the mixed water is discharged through a CSO. Detention tanks retain portion of stormwater mixed with wastewater that would otherwise directly enter the receiving water body, and they remove pollution by sedimentation. Accumulated water can be either discharged into watercourse, or redirected back to WWTP, after the rainfall event has ended. Retention tanks on combined network can be built either as in-stream, or off-stream [10]. Wastewater flows through the in-stream tanks even during dry weather, and when it starts raining, the tank begins to fill up. It helps alleviate the outflow hydrograph peak and retains part of the solid particles. Off-stream tanks are designed to retain pollution from the first minutes of rainfall event, termed as first flush. Both types of tanks have overflow weir crest, and after the capacity of the tank is reached, the tank overflows into the receiving waterbody. After the rain has ended, the accumulated water can be either pumped to WWTP, or discharged into the receiving waterbody.

Retention tanks on combined sewage network can have additional treatment elements, such as screens, brush screens, screw screens, flow separators, submerged vertical plates or UV disinfection.

2.3 Decentralised systems

2.3.1 Wastewater treatment

Decentralised wastewater treatment is characterised by managing wastewater close to its origins, which eliminates the need for large sewage networks. Decentralisation of wastewater treatment presents a scaling transition from enormous WWTPs to satellite treatment plants, semi-centralised supply and treatment systems, great block recycle systems, and cluster and individual on-site systems. [46] It is worth emphasizing that decentralisation does not always equal small, and decentralised systems do not have an upper limit of population equivalent or litres treated per day. [29]

From environmental point of view, decentralised solutions were often seen as insufficient in terms of treatment efficiency, mainly due to their poor design. The usual and most common form of decentralised wastewater management used to be a septic tank with the effluent discharging straight into soil, which led to many hygienic and environmental problems. However, with the introduction of some major technological developments such as filtration through natural soils, online monitoring, real-time alarm systems or programmable controls, on-site (decentralised) systems have become competitive with traditional centralised solutions. [90]

Generally, decentralised solutions are not perceived as economical when there is an existing connection to a central WWTP. In addition, centralised management of wastewater is still considered more cost-effective for large agglomerations, due to economies of scale. The slow shift from centralised to decentralised approach should consider these factors, and the choice of wastewater treatment technology should always follow environmental, economic and social criteria to comply with the 'fit for purpose' principle [92].

There are numerous types of decentralised wastewater treatment strategies. In general, decentralised systems can be categorized in three main categories, according to Orth [62]: a) simple sanitation systems, b) small-scale mechanical-biological treatment plants, and c) recycling systems.

SIMPLE SANITATION SYSTEMS

Simple sanitation systems are designed to retain the solid part of human-generated waste, while the liquid portion is usually discharged into ground. Usually, the main purpose is to protect

residents from pathogen contamination, and environment protection is of secondary concern, or is not considered at all.

These technologies include latrines, composting toilets, pour-flush toilets, aqua privies, pour-flush toilets, two-chamber composting tanks and others. They are cheap to implement, simple to construct and do not require external energy source, making them a suitable technology for developing countries [105]. However, they only offer basic sanitation level, therefore they are not suitable for European conditions and they would not comply with the European Union legislative framework [37].

SMALL-SCALE MECHANICAL-BIOLOGICAL TREATMENT PLANTS

Small-scale onsite systems treat wastewater from individual households, multiple houses, residential areas, institutions, or establishments. There are many technological options for decentralised treatment, and the choice of the optimal technology must always consider local situation, evaluate both construction and operational costs, take into account maintenance requirements, available space, level of wastewater pollution, required quality of effluent and possibilities of water reuse or nutrient recycling, expected life-span and many other factors.

For rural and less densely inhabited areas where space is not the limiting factor, extensive (low-tech) systems that use natural-like processes, rely on gravitational flow, can operate without any energy inputs, and only need occasional maintenance can be optimal. The disadvantages of these systems are the vast space requirements and the dependency on weather conditions such as temperature, sun irradiation or rain frequency. Extensive systems include wastewater stabilisation ponds and constructed wetlands.

For areas where space is limited, intensive (high-tech) systems can be suitable, as they are designed to occupy small footprint. Usually, the retention times are shorter, but the system needs frequent expert maintenance and external energy source for pumping, aeration, mixing etc, thus the operational costs are higher. Intensive systems include anaerobic baffled reactors, anaerobic filters, upflow anaerobic sludge blanket reactors or membrane reactors.

Septic tank (ST) is an underground sedimentation tank for primary treatment of wastewater on household or community scale. Septic tanks typically remove over 50% of influent TSS, and 30-40% of influent BOD [91].

ST is a multi-chamber reservoir made of precast concrete, fibreglass, PVC, or plastic. ST must have at least two chambers to minimize hydraulic short-circuiting. Most of the settleable solids are removed in the first chamber. The settled-down solids accumulate at the bottom of the tank where

they are degraded under anaerobic conditions which significantly reduces the volume of sludge [20], [91]. When the accumulated sludge occupies one to two thirds of total water depth, it needs to be removed, usually by vacuum trucks.

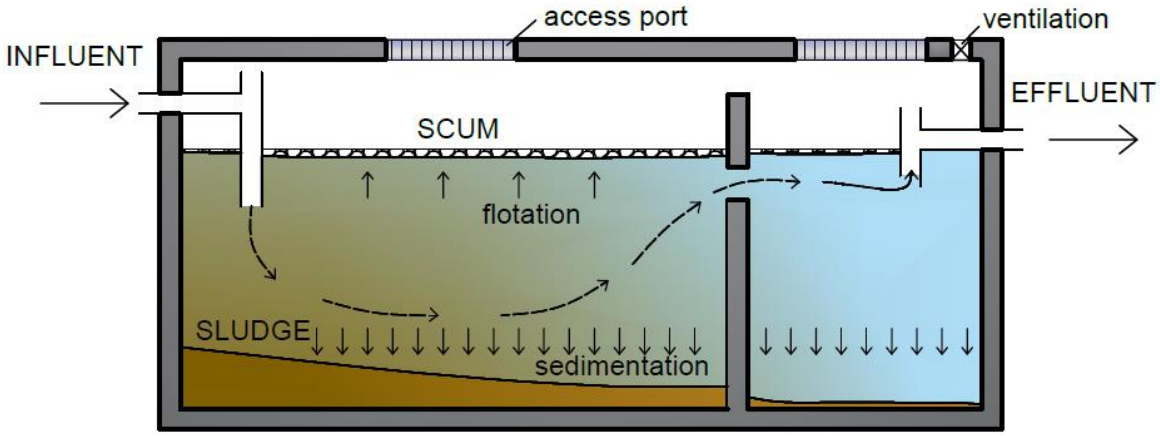

Figure 2. Schematic of a septic tank (adapted from Tilley et al. [91]*)*

The inlet and outlet pipes are T-shaped, with the lower arm submerged below the water level. This construction ensures oils, grease and other floating particles are captured in the tank [91]. Integral part of a septic tank is a ventilation system to allow the gases to escape. All chambers of the tank must be accessible for maintenance through access points.

Septic tanks can be followed by variety of decentralised secondary treatment technologies described below, or they can be connected to a sewage network and the pre-treated wastewater undergoes secondary treatment in a centralised WWTP. The risk of pipe blockage due to solids sedimentation is reduced, therefore pipes with smaller diameters and lower gradients can be constructed, which reduces the costs of piping and excavations.

The air-tight modification of septic tank is called **biogas settler**. It allows the possibility is to capture the biogas and use it as a biofuel, where feasible, mostly at community or institutional level. Captured biogas can be used directly as replacement of natural gas, transformed into heat, or converted to combined heat and power in a cogeneration unit. [91]

Septic tanks can be paired with a variety of **soil absorption systems** (SASs). SASs provide secondary treatment of wastewater by infiltration and percolation through an underlying unsaturated soil layer. Over time, a microbial layer forms and biodegrades the suspended particles in the effluent. Some particles are also removed by adhesion to soil particles [15]. In soil filters, part of the water

moves upwards and is absorbed by roots. Infiltration of treated wastewater is allowed only in areas with low groundwater level, low bedrock levels and permeable soils.

Leach field, also called drainfield, is a typical example of a SAS. Using a series of perforated pipes laid in trenches, the wastewater is distributed to the soil, where it percolates through unsaturated layer. Leach fields are usually classified as gravel (aggregate-laden) or gravelless (aggregate-free) systems [95]. In gravel systems, piping system is buried in a layer of gravel, underlined by a layer of sand. To prevent influx of small particles that could clog the gravel and roots from penetrating the pipes from above, geotextile fabric is used [80]. The pipes have to be at least 15 cm underground to prevent surfacing of the effluent [95]. At the surface there is a soil layer planted with grass. Trees and deep-rooted plants are to be avoided due to the risk of penetrating the geotextile fabric.

However, gravel fill can represent an obstruction to infiltration, and the use of gravel can impose a risk of soil compaction. Gravelless systems do not require aggregate fill which improves the overall infiltration capacity and allows better use of the surrounding soil, thus lowering the area requirements and enabling use even for steeper slopes [80], [95]. Leaching chambers are used in the most common gravelless setup. The chamber is a high-capacity, bottomless chamber usually made of plastic. It allows infiltration through the open bottom and through openings in the sides of the tank, and it can provide temporal storage.

The delivery of water to SAS can be either gravity driven, semi-continuous flow that is randomly distributed over time and dependant on trends in wastewater production, or it can be controlled by additional devices. Using pumps or siphons, intermittent dosing of wastewater is possible for better distribution of the flow. Using pumps, the distribution network can be designed as a pressurised system, which further improves the flow distribution.

Anaerobic baffled reactor (ABR) has similar treatment mechanics as septic tank, but the flow of water is directed by series of baffles that force the water to flow down due to the pressure head at influent point. The baffles divide the reactor in multiple compartments. The first part of the reactor is a settling chamber for solids sedimentation, followed by a series of upflow chambers. The wastewater enters the upflow chamber at the bottom and flows up, where is again redirected to the bottom of next compartment. Settleable solids accumulate at the bottom, so the wastewater is forced to flow through the layer of sludge. Consequently, the contact time of wastewater with sludge is increased, and the BOD removal can be as high as 90%. However, further treatment is still required due to the high number of nutrients and pathogens in the effluent. [91]

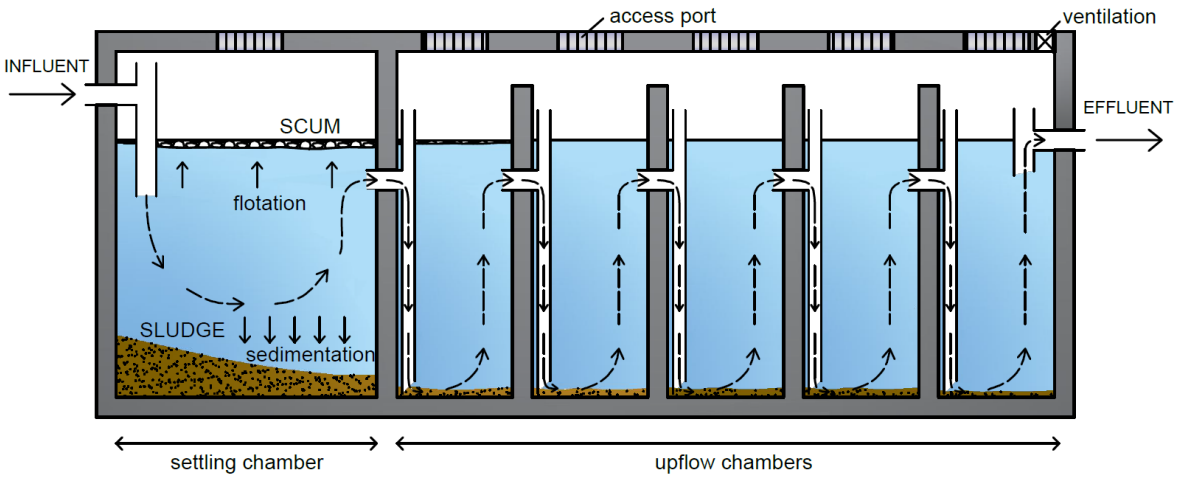

Figure 3. Schematic of an ABR (adapted from Tilley et al. [91]*)*

This design also favours removal of less biodegradable particles in the latter compartments, because all the easily degradable particles are consumed by bacteria in the front compartments [75], [91]. Like septic tanks, ARBs need to be properly vented, if biogas is not recovered for further use.

Anaerobic filter is a multi-chamber reactor that provides secondary treatment by forcing the wastewater to flow through multiple layers of submerged media covered in biofilm. The reactor is divided into settling chamber for solids removal and multiple anaerobic filter units where water is forced to flow upwards using a series of baffles. Filter consists of filter support grid and filter media of different sizes - the largest particles are at the bottom, the smallest at the top of the filter. The large surface area of the filter media allows quick biodegradation of incoming wastewater. Anaerobic bacteria colonise the filter and create biofilm that degrades organic particles in wastewater to create CO_2, CH_4 and heat. Anaerobic filter offers the possibility of biogas recovery, if the biogas is not collected, the filter must be vented to allow the gases to escape.

The removal of nutrients is insufficient and before discharge to surface waterbodies, further treatment is required. [91]

Primary treatment is necessary to avoid clogging of the filter, either in a septic tank, settler, or in a settling chamber preceding the filter units on household scale. On semi-centralised scale, anaerobic filters can be also combined with ABRs [91].

Maintenance includes removing sludge from the sedimentation chamber and cleaning the clogged biofilters. Biofilter can be declogged by backwashing (reversing the direction in which water flows) or by removing and cleaning the filter media.

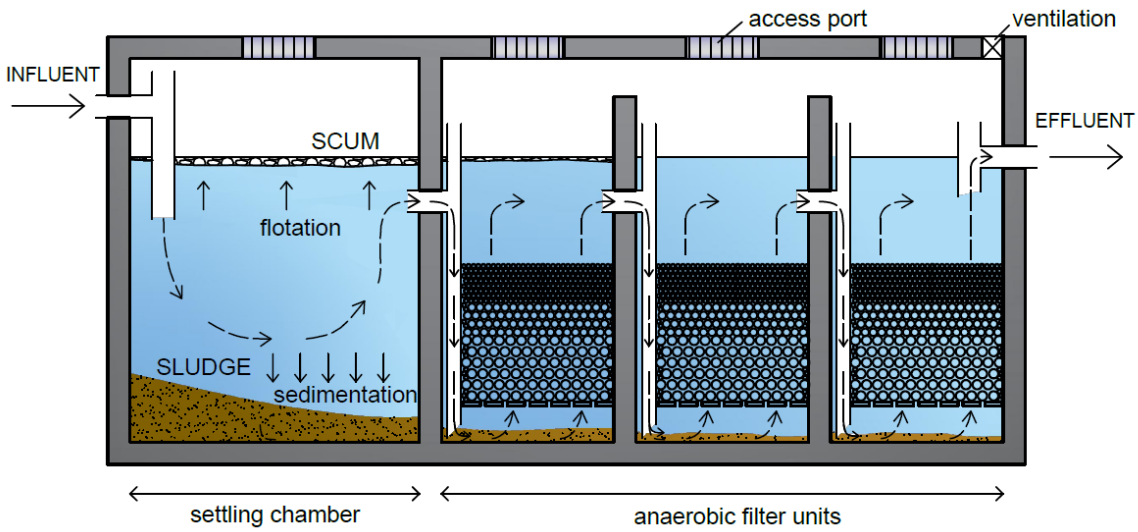

Figure 4. Schematics of an anaerobic filter (adapted from Tilley et al. [91] *)*

Upflow anaerobic sludge blanket (UASB) reactor is a community-scale technology that provides primary and secondary treatment under anaerobic conditions. The water enters the reactor at the bottom and flows upwards. Over time, suspended solids and microorganisms form a suspended sludge blanket. Particles from wastewater are biodegraded by the activity of anaerobic bacteria in the blanket and by mechanical filtering through the blanket. Bacteria-produced gas together with the upflow provide sufficient mixing in the tank.

The settling velocity of sludge granules and upflow velocity must be equal to maintain a stable position of the blanket, and the prevent escaping of particles from the sludge to the final effluent. In addition to that, the reactor has sloped walls at the top and usually there are baffles around the outflow zone to prevent escaping of sludge particles. Clarified water leaves the tank at the top.

Compared to aerated tanks, the energy requirement of an UASB reactor are much lower. Produced biogas can be captured and used as a biofuel or converted to energy.

UASB reactors show very good efficiencies of removing BOD, nevertheless, nutrient removal is low, therefore further treatment may be required to comply with the effluent quality regulations.

Expanded granular sludge bed (EGSB) reactors have higher height to diameter ratios and the upflow velocities are higher than in a UASB reactor. The sludge blanket expands so that the reactor space is completely mixed, as opposed to UASB reactors where occurrence of dead zones reduces the treatment efficiency. Expansion of the sludge bed allows better contact of wastewater with the organisms in sludge, and together with internal recirculation of effluent it contributes to higher efficiencies of removal of soluble particles. However, due to the higher velocities, more suspended

particles are carried away in the effluent stream. This reactor can accommodate wastewaters rich in COD [75], which lead to increased production of biogas. Rising biogas bubbles contribute to even better mixing.

Membrane bioreactors (MBRs) are high-tech treatment systems capable of producing effluent of high quality that is free of pathogens and organic substances. First, the wastewater is biologically treated in a suspended growth reactor, then it is filtered through a membrane. The semi-permeable membrane only lets through specific compounds, while solid particles are forced to remain in the tank. Microfiltration of ultrafiltration eliminates the need for secondary sedimentation tanks, which saves space and allows greater hydraulic loading. As such, they are suitable for densely populated urban areas where space is limited. MBRs show good efficiencies of COD removal, and depending on the pore size of the membrane, they can even capture micropollutant particles and pathogens. [11]

Although membranes are getting less expensive as their production increases from year to year, this technology has still higher initial and maintenance costs compared to more conventional solutions. Expert supervision is usually needed for maintenance. Moreover, the membranes need to be periodically cleaned with chemicals and they have limited lifespan which results in additional costs for chemical and replacement of the membranes. Rapid membrane fouling can be mitigated by pre-treating the water to remove oils (e.g. by dissolved air flotation). [11], [52]

After filtration, the water is stored in clean water tanks for later reuse (toilet flushing or even hand washing). To prevent recurrent growth of microorganisms in the storage tank, electrolysis, ozone, UV, or activated carbon treatment can be used.

Wastewater stabilisation ponds (WSPs) can provide primary, secondary, and tertiary treatment, depending on the setup, and they can produce high quality effluent while requiring minimum maintenance. The ponds are constructed in series with following layout: a) anaerobic pond, b) facultative pond, and c) aerobic pond. Natural physical, biological, and chemical processes are utilised to clean wastewater without any further energy requirements. However, their main disadvantage for application in densely inhabited areas is large footprint, and their dependence on temperature. As such, they are more suitable for warm climates. [78]

Anaerobic pond is the first stage of WSPs series that provides mostly primary treatment. In anaerobic pond, settleable solids are removed by sedimentation and subsequently digested by anaerobic bacteria, which reduce the sludge volume and stabilise it. Anaerobic ponds are the deepest (2 to 5 meters), but have the shortest retention times (one day to one week). The rate of

sludge removal depends on the influent TSS concentration, and on the rate of anaerobic degradation in the pond.

Facultative ponds combine anaerobic zone at the bottom with an overlying aerobic zone. In the bottom anaerobic zone, similar processes as in anaerobic pond take place. In the aerobic zone, further BOD removal due to the activity of aerobic organisms is ensured. The depth varies from 1 to 2,5 meters [77]. The detention time is 5 to 30 days [77]. Due to the photosynthesis cycle, the amount of oxygen in facultative ponds is variable, and hence the anaerobic zones are getting greater as the oxygen is consumed by algae at night, and smaller as algae produce oxygen during the day. When the pH rises due to microbial activity in the aerobic layer, die-off of pathogen occurs.

The main purpose of an aerobic pond is pathogen removal. To ensure aerobic conditions everywhere in the pond, they are the shallowest of the three types mentioned above (0,5 to 1,5 meters), so that the sun can reach even the deepest point to allow production of oxygen during photosynthesis everywhere in the pond. During the day, the bacteria consume CO_2, which causes the pH to rise high enough to ensure pathogen die-off. The tanks are designed for detention time of 15 to 20 days. It is also possible to cultivate fish and plants that consume nutrients to enhance nitrogen and phosphorus removal. Aerobic ponds can be designed also in series to provide higher effluent quality.

For less polluted waters such as greywater it is also possible to omit the anaerobic pond, and combine only facultative with aerobic ponds.

All the ponds must have a liner to prevent infiltration of wastewater directly to groundwater, and they should also be fenced to prevent people and animals from coming into contact with the pathogens in wastewater.

Septic tank or other solids removing technology can be followed by a free surface, horizontal or vertical flow **constructed wetland** (CW). Constructed wetlands are cheap to implement and to operate, and they not require intensive maintenance. Pollutants are removed by microbial activity and nutrient uptake by vegetation roots, together with sedimentation and adhesion to the vegetation. CWs have to be combined with septic tanks or some other form of sedimentation pre-treatment to clogging, and also to lower the BOD. [11]

Vegetation with extensive root networks is preferred for CWs, because roots not only maintain the permeability of the substrate, but also provide small oxygen zones around the root hairs in the otherwise anaerobic environment below the ground. Plants also bring amenity to the area, and provide wildlife habitats. After dying, plants fall at the ground where they decompose and provide

insulation, or are removed. CWs usually have low energy requirements, but on the other hand have a large physical footprint. The treatment efficiencies are better with higher temperatures.

In **vertical flow CW**, water is dosed from above in multiple batches a day, percolates through the filter media planted with vegetation and is drained using a set of pipes in the bottom. Intermittent dosage of wastewater ensures alternation of aerobic and aerobic conditions in the filter. It also introduces temporal starvation phases for the microbial organisms, which prevents them from rapid growing. The vegetation roots help to maintain permeability of the filter, and introduce small aerobic zones close to the root zone.

Horizontal flow CW are large gravel- and sand-filled beds, planted with vegetation that has deep roots and can survive the wet environment. The influent is distributed in the wide inlet zone and flows horizontally under the ground to the outlet zone. Conditions in the wetland are aerobic, therefore the gravel bed should not be very deep to allow the oxygen to reach the bottom. The particles from wastewater are removed by biodegradation, adhesion to the roots and predatory activity of higher organisms. However, these systems do not provide nitrification due to the limited concentrations of oxygen.

In **free surface CW**, water flows above ground, and the vegetation is flooded up from 10 to 45 centimetres above ground. Emerging, submerged, and floating plants can be present. Unlike in horizontal flow CWs, water is exposed to sunlight and other atmospheric conditions. Various biological, physical, and chemical processes take place. Compared to subsurface flow CWs, both horizontal and vertical flow, the physical footprint of this measure is greater. Because of the risk of human contact with contaminated water is high due to the open surface, this type of CWs is rarely used for secondary treatment.

These three types of CWs can be variously combined in a **hybrid constructed wetland**. Hybrid configurations usually combine vertical and horizontal flow CWs. As mentioned, horizontal flow CWs are not suitable for nitrification of ammonia due to insufficient oxygen content, whereas vertical flow CWs provide conditions for nitrification, but denitrification is not supported.

An example of a hybrid CW system is the two-stage horizontal-vertical-flow CW. After primary treatment, the water flows through a horizontal flow CW where majority of TSS and BOD are removed [99]. After this step the water is intermittently dosed into a vertical flow CW where the nitrification bacteria thrive. The effluent can be then pumped back to the horizontal flow CW to achieve denitrification, or can be discharged. In general, hybrid systems display better efficiencies, but are more complex and expensive to operate.

FINAL DISPOSAL OF SLUDGE FROM ONSITE SYSTEMS

Sludge from septic tanks, biogas settlers and ABRs needs to be periodically removed to maintain the proper functioning of these systems, and to prevent the settled solids to be carried away.

Prior to final disposal, efforts are made to reduce the volume of sludge to be treated and transported and for sludge stabilisation and hygienisation. Sludge is stabilised when there no more on-going biological processes. Hygienisation of sludge ensures die-off of all the pathogens.

Sludge that has been stored under anaerobic conditions for longer period is usually partially stabilised, hence it can be dewatered more easily. Although the sludge volume is reduced in septic tanks and other anaerobic reactors, it contains pathogenic organisms, and as such represents a health risk to the community. [41]

Sludge from household and community systems can be transported to a (semi-)centralised WWTP to be treated in anaerobic digesters to stabilise the sludge and reduce its volume, the final product is then applied to agricultural land as fertiliser. In the past it was acceptable to landfill or incinerate thickened and dewatered sludge, but this approach now discouraged by authorities due to insufficient sustainability.

Anaerobic digestion is already widely used in centralised systems to digest primary and secondary sludge; however, it is now considered a viable alternative also for decentralised and semi-centralised systems. It does not require high energy inputs and the footprint of anaerobic digesters is relatively small, allowing their installation even in highly urbanised areas. The produced biogas can be captured and used as energy source, which can help offset the operational costs and carbon footprint of the process.

The sludge production is 3 – 20 times lower, and the sludge can be dewatered more easily as compared to aerobic systems [21], which favours the consequent use of the digestate as a fertiliser.

Composting is process suitable for sludge treatment both at household and semi-centralised level. The final product is pathogen-free, stabilised and suitable for agricultural application as a fertiliser. Composting requires specific C/N ratios and moisture content for optimal efficiency of the process. For wastewater sludge the C/N ratio should be around 25, and the optimal solids content is in the range of 15 – 20% [60]. To adjust the C/N ratio and solids content to required values, bulking agents such as cow dung, fruit and vegetable waste, grass clippings etc. are added. Bulking agents also contribute to better oxygen distribution in the composted material. To ensure sufficient aeration, compost in household composting facilities should be turned which requires manpower. During the degradation process, high temperatures occur due to microbial respiration, which results in

pathogen die-off. To contain the heat inside of the pile, sufficient insulation is necessary. Compost is considered sanitised when temperatures are 50°C and above for one week or more. [105]

Vermi-composting is a suitable process for sludge stabilisation during which the material is digested by earthworms. The sludge must be mixed with other materials to provide favourable living conditions for the earthworms. [76]

In large-scale plants, thickening and dewatering is usually achieved using mechanical devices, but at community scale it is feasible to utilise natural processes. In general, dewatering and thickening significantly reduces the volumes of sludge for further treatment; however, it does not produce hygienically safe product. Another design consideration is the dependence of natural processes on climatic factors such as rainfall or average sunshine duration.

Sedimentation or thickening ponds have long retention times in the range of days of even weeks. They can be built in warm and temperate regions. Sludge settles down and is degraded under anaerobic conditions, while the supernatant water is pumped back to the beginning of treatment process. The thickened sludge can be composted of transported to a planted or unplanted drying bed.

Unplanted drying beds are reservoirs underlined with drainage systems for collecting the leachate. Part of the water is evaporated, other part percolates through the layer of gravel and sand, and is collected by the drainage system and conveyed to be treated. There are two alternating cycles – feeding cycle and drying cycle. After each drying cycle the sludge must be removed, before new batch is applied.

Planted drying beds have similar construction as unplanted drying beds, but the vegetation contributes to additional dewatering due to evapotranspiration. Plants also take up nutrients from the sludge during their life cycle. While the sludge from unplanted beds needs to be removed after each drying cycle, in planted beds it is possible to dose the fresh sludge on top of the previous layer. Hence, desludging can be done only every 5 to 10 years, and the collected sludge is pathogen-free and suitable to be applied on agricultural land.

RECYCLING SYSTEMS AND SOURCE SEPARATION

Traditional wastewater management conveys wastewater from individual households to one central WWTP. To do this, large volumes of drinking-quality water are needed to transport the minor volumes of highly polluted blackwater. Mildly polluted water from showers and sinks gets mixed with urine and human faeces, resulting in quality degradation of mildly polluted greywater and dilution of the concentrated waste containing high loads of valuable nutrients.

This approach is unsustainable because it requires large volumes of relatively clean water to transport the minor fraction of blackwater. Due to transportation and treatment of large wastewater volumes, the overall costs of the system increase. Moreover, after diluting highly polluted blackwater with less polluted greywater, energy recovery from wastewater, recycling of nutrients and reuse of treated water become more difficult.

In general, there are two types of centralised drainage systems: either separated system that uses dual pipes for stormwater and wastewater, or combined system that conveys both stormwater and wastewater together. None of these two options are ideal in terms of sustainability. Combined systems overload the WWTP during rainfall events and the overflowing of Combined Sewer Overflows (CSOs) during heavy rainfall causes significant environmental stress to the receiving waterbody, not to mention the hygienic and visual aspects. Moreover, mixing wastewater with stormwater is not the optimal solution for the treatment plant, as these two types of urban water have different characteristics. Domestic sewage is the main source of organic and nutrient pollution, whereas stormwater contains high loads of heavy metals, concentrated especially in the initial volumes termed as 'first flush'. [21], [11]

Separated systems prevent the pollution caused by CSOs overflowing, nevertheless, the pollutants from wet and dry depositions are usually directly entering the receiving waterbody.

To avoid merging household wastewater streams with diverse characteristics and different levels of pollution, the source separation concept is now promoted as a promising future technology [11], [63]. The future of wastewater treatment anticipates local systems that reuse water and produce nutrients for further use, successively closing the water and nutrient cycles. [46]

Domestic wastewater streams can be divided into blackwater and greywater (*Figure 5*). Blackwater is wastewater containing waste from toilets, i.e. faeces and urine, plus water needed to flush the toilet. Blackwater can be further divided into yellow water (urine) and brownwater (faeces). In terms of volume, blackwater represents a very small fraction of domestic wastewater production, nevertheless, it contains majority of nutrients (N, P, K). According to Otterpohl, about 87 % of nitrogen, 50% of phosphorus and 54% of potassium comes from urine [64]. Brownwater is a major source of phosphorus (around 40%) and it represents almost half of total COD [64].

Greywater is mildly polluted water that has been used for personal hygiene, washing dishes, laundry etc. Its volume is about 45 to 180 times greater than volume of blackwater [64]. It does not contain high loads of nutrients like blackwater and it should not represent a hygienic danger as it does not come directly in contact with human faeces.

Therefore, greywater does not require sophisticated treatment before it can be reused for various purposes such as irrigation, flushing the toilets or groundwater recharge. However, it can contain significant amounts of heavy metals, chemicals from detergents and cosmetic products, and grease and fats from kitchen sinks. The nitrogen content is almost negligible (3%). Greywater contains approximately one third of total potassium and one tenth of total phosphorus. The COD percentage is quite significant – around 41%. [64]

Source separation not only reduces the operating costs of wastewater treatment by reducing volumes to be treated, but it is also an important step in on-site water reuse, thus reducing consumption of potable water for purposes where water of non-potable quality would be sufficient. Separate collection of blackwater allows more straightforward recovery of nutrients, diminishing the need for manufacturing mineral fertilizers. In addition, blackwater contains majority of pathogens and micropollutants, thus by separate collecting, better control of contamination threats is possible. Moreover, concentrated volumes of blackwater can be used to produce sustainable energy for example in anaerobic reactors. [45]

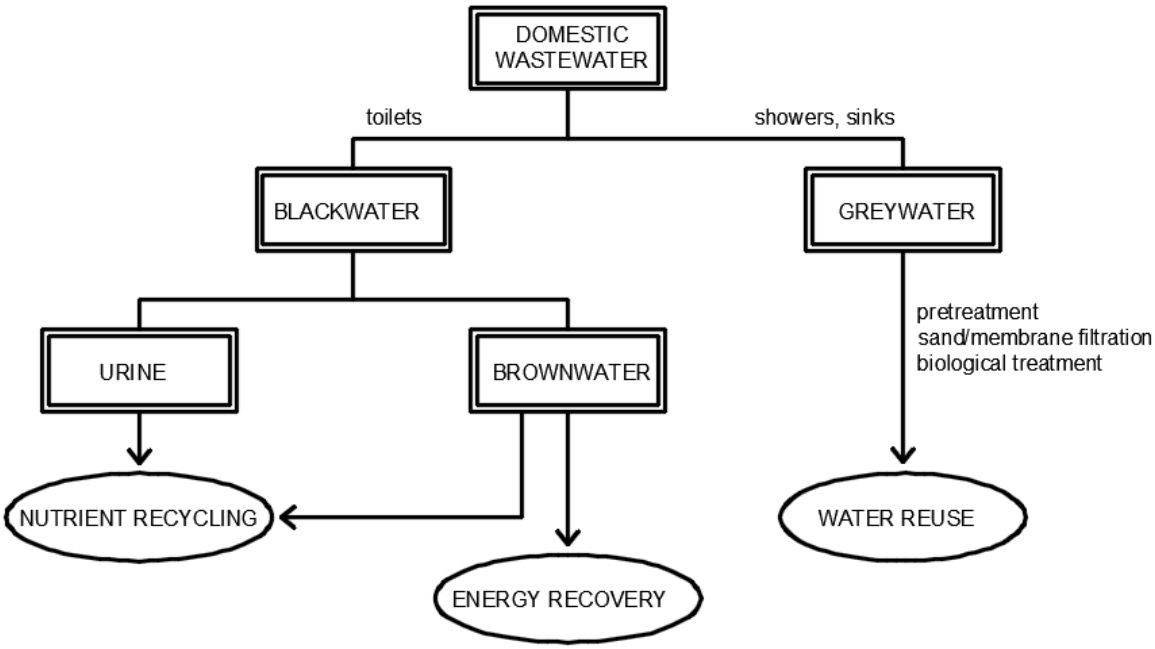

Figure 5. Schematics of domestic wastewater streams

The technical infrastructure system for **urine source separation** consists of user interface for urine collecting, conveyance, treatment and reuse technologies. [51]

Collecting systems are either urine-diverting toilets, or waterless urinals. The most modern urine-diverting toilets are equipped with sensors and two separate pipes. When the sensors detect urine flowing in the toilet, a valve is opened that allows the separate collecting of urine. After flush, the urine chamber closes, and the faecal material together with toilet paper and water used for flushing is transported to further processing using the second set of pipes.

Blackwater can be collected using vacuum toilets, which minimizes the amount of water used for flushing. However, currently there are no urine-diverting vacuum toilets available on the market.

The transport of urine, either to on-site treatment or to a central WWTP, is still a challenge from engineering point of view. Ureolytic bacteria degrade urea in urine, causing the sudden rise of pH. As a result, dissolved salts begin to crystallise, and precipitates begin to build up, especially struvite and calcium phosphates. The precipitation formation of salts is lessened when flushing water is added, but this in turn causes the need of larger volumes of urine storage reservoirs. The difficulties associated with urine transport contribute to even bigger appeal of decentralised solutions which do not rely on transportation.

Urine separation systems can be implemented also with traditional centralised systems. Separation and local storage of urine combined with existing sewage systems could help offset the morning surge of nutrient pollution coming to WWTP by temporal storage of urine and gradual distribution of the nitrogen coming to WWTP. This practice of “peak shaving” would lead to smaller nitrogen peaks and therefore the designed nutrient removal capacity of the WWTP could be reduced. In areas where WWTP capacity is already depleted, urine source separation could help evade further expansion. Also, by releasing urine only during dry weather, overflowing of CSOs would not have as severe environmental impacts, as approximately half of the nutrients comes from urine.

Another potential scenario is to release all urine at once at night, but this comes with a risk of an unpredicted rain event, possibly leading to contaminating receiving waterbodies with high loads of nutrients [44]. Another issue would be odour nuisances due to transporting only slightly diluted urine.

Nowadays, the urine can be treated for example using combination of biological nitrification and distillation [27]. This process aims at separating water from urine, instead of removing nutrients. The final product is still liquid, but the volume is significantly reduced, and there is almost no loss of nutrients due to processing.

Untreated urine contains high concentrations of ammonia, a volatile substance with a characteristic odour. To stabilise urine and to prevent ammonia from volatizing, bacterial activity is supported in

order to oxidise part of the ammonia to nitrate. This process is called nitrification. Ammonia is first converted to nitrite (NO_2^-) due to the activity of ammonia-oxidising bacteria, and nitrite-oxidising bacteria then produce nitrate (NO_3^-). With the production of nitrate, the pH gradually drops, and after half of ammonia is oxidised, the pH settles at 6.5 and the other half of ammonia stabilises itself as the non-volatile ammonium. [27]

To secure the steady operation of nitrification reactors, pH must be kept in within limited range and the urine supply must be balanced – if there is too much ammonia in the solution, ammonia-oxidising bacteria produce too much nitrite and accumulation of nitrite occurs, causing the pH to fall up to 5.5. At pH value 5.5, ammonia-oxidising bacteria are inhibited, and nitrite-oxidising bacteria start to break down accumulated nitrite, causing the pH to rise again. On the other hand, insufficient supply of urine can promote increased growth of acid-tolerant types of ammonia-oxidising bacteria, potentially lowering the pH up to 2. This results in die-off of the common ammonia- and nitrate-oxidising bacteria, resulting in destabilisation of the nitrification process. Therefore, process control is a very important part of operation of nitrification reactors.

Pilot-scale projects are currently operated in Switzerland and South Africa. These pilot projects utilize aeration columns with plastic carrier media. [27]

Stabilised urine is then stored and conveyed in batches to the distiller. Using distillation, the urine is separated into two parts – distilled water that contains less than 1 % of nitrogen from urine, and concentrated liquid fertiliser containing majority of all the nutrients. Pathogens are eliminated after exposure to the temperature of 80 °C for at least 30 minutes. [27]

Another possibility of urine treatment is electrolysis. This method utilizes electrodes of different materials and electrical current. However, compared to other methods, electrolysis is rather energy-intensive.

For areas without electricity and for developing countries, pilot-scale installation of struvite precipitation plant was installed in Nepal [27]. This method recovers only phosphorus, and the effluent still need some additional treatment before it can be safely discharged.

Probably the least demanding method of urine treatment is storage and consequent application to agricultural land. The minimum storage time needed for ensuring the hygienical harmlessness depends on temperature, pH, concentration of urine, intended use and on the scope of the system. For example, in single household systems, the risk of transmitting a disease via consumption of urine-fertilised crops is negligible compared to the probability of transfer between family members, provided at least a month will pass between fertilization and harvesting. According to the Swedish

guidelines, the recommended storage time is in the range of 1 to 6 months. After six months of storage is it safe to apply urine to all crops. [79] The storage should take place in a tightly sealed container to prevent risk of human or animal contact, to minimize evaporation and eliminate odour nuisances.

Brownwater is the most polluted stream of domestic wastewater. Similar technologies as for faecal sludge can be used to treat brownwater in decentralised systems, such as composting, drying, and anaerobic digestion. Reuse options are application to soil, landscaping, and energy recovery in the form of biogas.

Source-separated brownwater streams are usually only slightly diluted by water, which makes them optimal for anaerobic digestion because high biogas yields make the operation of the treatment facility more economic. Co-digesting of brownwater together with organic household waste can increase the methane yields even further, and it helps reduce the amount of waste that is landfilled of incinerated. Moreover, digestion under thermophilic conditions is recommended by WHO as an effective way of sanitizing brownwater. [79]

Aerobic composting can be implemented on a small scale, or in semi-centralised systems. To ensure that the compost will reach sufficient temperatures to kill the pathogens, faeces have to be mixed and co-composted with bulking material. [60]

Generally, before brownwater can be applied to agricultural land, it needs to be stabilised to ensure there are no ongoing fermentation processes, and pathogens must be eliminated in a hygienisation process. Unlike for urine, simple storage of brownwater at ambient temperature is not considered safe practice as faeces contain large numbers of pathogens. Over time, die-off of pathogenic organisms occurs due to unfavourable living conditions, but some resistant forms can survive in untreated brownwater for years. There are many factors that affect the die-off rates of various pathogens, and usually these factors are interrelated. Some of the most influencing factors are temperature, pH, ammonia, moisture, solar radiation, nutrients and others. [79]

Pathogen inactivation can be achieved using chemicals, such as ash or lime. This method causes pathogen die-off due to elevated pH (11-12). In addition, it helps reduce malodours, prevents flies from feeding off the faecal material and subsequent distribution of contaminated matter and reduces the moisture content. However, adding chemicals can have hindering effects on subsequent composting processes. Also, this method is not very well-suited for urban areas.

Recently, a new technology for faeces treatment is being modelled in Eawag, called hydrothermal oxidation. This technology is based on high temperatures (300 - 700 °C) and high pressures (24 – 44

MPa), causing the faeces to oxidise completely, using oxygen, air, and hydrogen peroxide as oxidants. The final products of this reaction are carbon dioxide, ammonia and water. [36]

Greywater, as the least polluted part of domestic wastewater, is the most suitable for water reuse after appropriate treatment. Treated greywater can be reused for irrigation, toilet flushing or even hand washing.

To produce effluent of high quality, membrane processes such as gravity driven membrane filtration or membrane bioreactors are now widely used. Where there is sufficient space and suitable climatic conditions, extensive technologies utilising natural-like treatment are also an option, such as constructed wetlands or wastewater stabilisation ponds. Depending on the required effluent quality, wetlands and stabilisation ponds can be followed by a tertiary disinfection step. [11]

2.3.2 Stormwater management

In recent years, various adverse impacts on the environment caused by existing drainage systems have been identified. As a result, sustainable urban drainage systems (SUDS) have been introduced as an alternative to the traditional drainage systems. SUDS are designed to offset the negative impacts of urban areas on natural hydrological cycle by returning the flow regime back to its natural form. [8]

Urban water management concepts are described in multiple guidance systems, such as best management practices (BMPs), low impact development (LID), water sensitive urban design (WSUD) and others. However, the objectives of these concepts are very similar to SUDS. [28]

SUDS aim to quantitively control the runoff volumes and rates as opposed to rapid transportation downstream. By allowing infiltration, groundwater recharge is supported. Other key goals of SUDS are enhancement and protection of water quality, and enhancing amenity and aesthetic aspect of urban areas, while preserving the natural characteristics of waterbodies. SUDS measures can provide habitats in urban areas, thus promoting biodiversity. Implementation of SUDS should also consider other environmental and social needs. [53]

Sustainable urban drainage planning implements a variety of solutions, which can be divided into two main groups: structural measures and non-structural measures. Structural measures are a set of technical devices and modifications of the drainage net intended to support more natural runoff conditions in the urban area. Non-structural measures incorporate the use of environmental

education, flood mapping, environmentally conscious urban planning and monitoring systems. [53] Following chapter briefly introduces the most common structural measures used in SUDS.

SOURCE CONTROL MEASURES

According to the Cambridge SUDS Design & Adoption Guide, source control is management of rainwater where it falls. In other words, source control measures are aimed at techniques that deal with stormwater as close to the source as possible, instead of transporting it elsewhere [106]. The idea of source control is to store the rainwater from frequent but less intense rainfall events. This approach results in decreased surface runoff volumes and rates, reduces the risk of flooding, and improves water quality by removing solids and pollutants. Infiltration of stormwater also promotes groundwater recharge and increases baseflow.

Source control measures are often owned by private subjects [106]. It is desirable for the local city council to support implementing these measures by introducing incentive schemes, because by reducing runoff volumes and removing pollutants, the operation and service life of infiltration devices, swales, conveyance channels, retention basins and detention basins ponds can be significantly longer, resulting in reduced maintenance and repair costs.

Rainwater harvesting is a source control measure which can be implemented at different scales, from small water butts to rooftop reservoirs and large collection tanks. Major benefit of rainwater harvesting and reuse is reduction of potable water consumption for activities where rainwater provides sufficient quality, such as garden watering etc.

Green roofs are a SUDS technology used to reduce surface runoff volumes and peaks, remove pollutant loads and bring amenity to the area. They can also contribute to better thermal and sound isolation of the roof. There are two types of green roofs, intensive and extensive.

Intensive green roofs are usually found on commercial buildings, such as office buildings or shopping centres. They have a thicker layer of substrate which can be planted with grass, flowers, brushes and even trees. There can be pathways that allow walking and other features commonly found in parks, e.g. benches, tables, ponds, fountains etc. The roof consists of a layer of growing medium, underlined by fleece or any other material preventing roots from penetrating the underlying structure. Below the soil layer there is usually some type of drainage, followed by insulation layer. Between all these components and structural construction of the roof there must always be a waterproof membrane [14]. Intensive roofs require higher maintenance compared to extensive roofs, such as mowing the lawn, clipping, and pruning the plants or irrigation in dry periods.

Figure 6. An intensive green roof in Vítkovice, Czech Republic (1st place in the Green Roof 2016 Award) [109]

Figure 7. An example of extensive green roof - British Horse Society Headquarters [104]

Extensive roofs require thinner layer of the planting medium as compared to intensive roofs. Typically used types of vegetation are grasses and succulent plants, which are capable of surviving longer dry periods in case of no rainfall event. They are not irrigated and without rain, they can transiently turn brown. They are usually found on residential buildings. Unlike extensive roofs, they are not made for walking, although they can be accessed for maintenance. Because the additional load on the roof is not as heavy as with extensive roofs, in some cases they can be installed on existing roofs.

Permeable pavements or other pervious surfaces can be used in driveways, pavements, parking areas and some roads. The surface is underlined by a layer of gravel, which serves as temporal storage of water, before it either infiltrates the ground, or it is diverted towards another SUDS measure. The most common surfacing materials are concrete or clay tiles. Tiles allow the water to pass through the spaces between them. Other surfaces, such as porous asphalt, reinforced grass, or gravel, allow infiltration across the entire surface.

The use of permeable pavements is limited to areas where stormwater does not contain excessive loads of solids which could clog infiltration passages. In case of clogging, permeable pavements can be unblocked using water jets and suction. [106]

Figure 8. Permeable pavement [74]

Some devices mentioned below in other categories can be also classified as source control measures, for example **swales, filter strips, soakaways** and **rain gardens**. [14]

DETENTION BASINS, RETENTION PONDS AND WETLANDS

These types of measures attenuate peak flow by detaining water for a period termed residence time. Additional benefits can arise from pollutant removal by sedimentation, eventually bacterial decomposition. In general, they are rather space-intensive. They can have additional amenity and diversity benefits. The volume of stormwater is usually not reduced, although some infiltration may be supported where there is no risk of contaminating groundwater. [14]

Retention ponds and reservoirs provide attenuation and treatment of stormwater. Pollutants are removed by sedimentation, biological activity, and nutrient uptake by aquatic vegetation. Both emergent and submerged types of aquatic vegetation are present, providing habitats and serving as an aesthetic landscaping element. However, standing water poses some health risks and can promote proliferation of mosquitos. Suitable sites for constructing a retention pond include parks and public areas. The footprint of this measure is rather big; thus, the implementation is not feasible in densely populated areas.

Figure 9. Retention pond [14]

Maintenance comprises of sediment and debris removal, vegetation maintenance and eventual repairs of eroded banks.

Ponds can also utilize infiltration, if there is no risk of groundwater contamination. In case of possible groundwater contamination, they must be constructed with liner. [43]

Detention basins do not have a stable pool of water, unlike retention ponds. Water is detained after a rainfall event and discharged slowly to reduce the peak flow. Some particles are removed by sedimentation, although the residence time is usually not long enough for fine silts and clay to settle down. Bacterial activity and soil absorption can improve the quality even further. Necessary maintenance involves debris and litter removal and maintenance of vegetation.

Figure 10. A roundabout with a detention basin [100]

Wetlands are shallow water bodies with very dense vegetation cover, designed to attenuate and treat the stormwater. They are beneficial in terms of amenity and biodiversity. They usually comprise of:

- inlet zone, where the coarse sediments are removed by sedimentation
- macrophyte zone, where the water quality is improved by aerobic decomposition, adhesion to vegetation, and enhanced sedimentation of fine particles
- bypass channel with high flow to protect the macrophyte zone

Ideally, constructed wetland should be paired with another SUDS component located upstream to prevent silting and to ensure equalized flow over time. The residence time in wetland is usually around three days. [14]

Figure 11. Constructed wetland [59]

To preserve aerobic conditions in the wetland, shallow depth must be maintained even during rain event, which limits the flow attenuation. The design should prevent mosquitos proliferation, e.g. avoid stagnation of water and allow access for natural mosquitos predators. [87]

Maintenance includes removing litter and debris from the inlet zone. Vegetation must be maintained to ensure sufficient treatment; regular checks are needed to ensure noxious plants will not take over the desired species. Dead plants need to be removed and replanted. Sediment should be prevented from entering the system, monitoring and mining the sediment from the inlet zone is essential. [87]

CONVEYANCE SYSTEMS

Conveyance of surface water runoff in SUDS relies on surface channels to promote attenuation and treatment. Traditional underground pipe systems are not perceived as sustainable.

One of the device used for conveyance of stormwater is a **swale**. Swales are shallow and broad grass-lined channels. They are designed to store and/or convey stormwater. Moreover, they remove some pollutants through filtration, sedimentation, adsorption on vegetation roots, and microbiological breakdown. They reduce both runoff rates and runoff volumes (where infiltration is possible).

To decrease the slope of the swale and to promote infiltration and sedimentation, check dams can be built. Swale can be equipped with underdrain consisting of perforated pipe and sand or gravel bed. Swales are often combined with filter strips to prevent clogging. [106]

Figure 12. A conveyance swale in Dundee, Scotland [100]

Stormwater can be transported also via **channels and rills** with hard edges. Planting channels with vegetation is encouraged, where possible, to improve pollutants removal and to increase amenity. The main purpose of these SUDS measures is to collect stormwater and slow it down, but they can also remove solids by sedimentation and adsorption on roots, improving the efficiency of latter stages of SUDS measures.

INFILTRATION SYSTEMS

Infiltration devices decrease surface runoff by allowing part of it to infiltrate. The dimensions of these measures depend on the permeability of the soil they are built upon, therefore implementation is only reasonable in areas with permeable subsoils. Inflitration is not suitable in areas with high groundwater level and on contaminated sites. Another design consideration is the distance of infiltration device from surrounding buildings. [14]

Infiltration systems are prone to colmatation, therefore they should be combined with filter strip or paired with another filtration device for the removal of fine soils.

Infiltration trenches are linear structures that are designed for temporal storage of stormwater and infiltration. The trench is excavated and filled with coarse rock secured with porous membrane, infiltration takes place at the bottom and sides of the trench. Reduction in both runoff rates and volumes is accomplished due to infiltration. Removal of pollutants is achieved through filtration, adsorption and biochemical activity of the microorganisms present on the fill or in the soil.

This measure is not suitable for sites with high influx of fine silts and clay, as it is prone to clogging. Some level of pretreatment is also desirable. [14]

A modification of an infiltration trench is a **waste filter**. It has the same function as infiltration trenches, but it is equipped with perforated pipe for conveying the water from the collection area to the final point. [69]

Infiltration basins are lowered grassed areas. During a rainfall event, surface runoff from surrounding areas flows into the basin, where it slowly infiltrates, thus reducing surface runoff volumes. Pollutants are removed due to soil filtering. Often some level of pretreatment is needed to prevent colmatation.

This measure has quite large footprint, and it can only be implemented in areas with permeable soils, without risk of groundwater contamination. [14]

Soakaways, or infiltration galleries, are linear structures intended for infiltration. They are excavated in the ground, and either lined with porous material (bricks, pre-cast concrete, polyethylene ring or perforated storage structures), or filled with crushed rock to support the sides and to prevent them from collapsing. The bottom of the soakaway is covered in a layer or sand and gravel for flow dispersion. Soakaways can be combined with other measures that remove solids in order to extend operational life. When the infiltration rates decrease due to colmatation, it is possible to remove the filling material and refill the soakaway.

Soakaways are equipped with removable concrete lids to prevent injuries. They can be implemented adjacent to parking lots and roads, but there should be a distance between the road and the soakaway to avoid unsolicited compaction of soil. [22]

Rain gardens are relatively small devices for infiltration of water flowing from non-polluted surfaces such as roofs. Rain gardens are local depressions set up with plants, which can be installed on private properties for management of roof runoff, reducing the volume of stormwater entering the drainage system. The maintenance is rather simple, plus they can represent a pleasing visual aspect.

The limitating factor of implementation is the slope of surrounding terrain. It is necessary to prevent clogging in order to ensure proper functioning. [14]

Figure 13. A rain garden [88]

FILTRATION SYSTEMS

Filtration systems provide treatment of stormwater by trapping solid particles within the soil, on roots, plants or on geotextile layer. They help extend the operational life of other SUDS components, especially infiltration devices. However, to prevent clogging of filtration systems, excessive loads of fine clays must be prevented, e.g. by landscape management.

The main purpose of **filter strips** is to remove solid pollutants using filtration and sedimentation. Filter strips, or grassed strips, also serve to slow down the runoff rate and partially infiltrate a portion of runoff volume. They are relatively economic to implement, easy to maintain and can represent an aesthetic element in urban areas. They are usually implemented adjacent to parking lots and other impermeable areas, and they have a small slope towards another SUDS measure or receiving waterbody.

Filter trenches are shallow linear structures filled with rubble. They treat inflowing stormwater, transport it and can provide temporal storage. The filter material is prone to blockages due to excessive influx of fine particle soils.

Bioretention areas are vegetated landscape elements for treatment and storage of stormwater. They are underlined with drains to convey the stormwater away. They remove pollutants, and reduce runoff volumes and rates. [14]

2.4 Determination of optimal degree of centralization

Until recently, centralised solutions were strongly preferred as they were believed to be the safer and more economically viable solution in all cases no matter the terrain setting or population density. [25] Construction of the water management infrastructure network requires significant investments, therefore the endeavour in the past was to build as large WWTPs as possible to maximise the served area and to lessen the price per treated unit of volume. This inevitably led to even further increased centralization.

It was not until 1960 when it became apparent this approach is not sustainable from a long-term point of view due to a number of reasons described below. [53]

First, due to the increasing urbanization in developed countries, many of the existing plants are at their maximum capacity, and often there is no space for expansion anymore due to new development. Expanding the existing sewage networks would also bring many complications as the sewage network often has the same routes as transport network.

Moreover, traditional conveyance-based systems require large quantities of mildly polluted water from showers and sinks to transport a small volume of highly polluted water from toilets, and in combined sewage systems mixing of wastewater with stormwater occurs. This makes treatment more difficult and expensive, and limits the potential reuse of reclaimed wastewater. In addition, transporting large volumes of wastewater increases energy consumption, e.g. for pumping.

Wastewater contains some valuable resources, such as water or nutrients. In large-scale WWTPs, the opportunities for water reuse are limited, and further transport of reclaimed water would impose an additional financial burden. Traditional wastewater treatment is more oriented on biological nutrient removal, rather than on nutrient recovery. [29]

It remains a fact that the failure rate of on-site systems is many times higher than failure rate of large centralised WWTPs. According to US EPA, 10 to 20 percent of septic systems fail at some point of their operational lifespan. The most common causes of failures include improper installation,

such as deficient design, siting, wrong choice of technology or faulty construction; operational failures and failures due to insufficient maintenance. [94]

Failures and malfunctions of large WWTPs are rare compared to smaller on-site systems, however, the consequences are usually far-reaching. Decentralised solutions represent distributed risks across urbanized areas, whereas centralised WWTPs impose risks on a larger scale. [6], [19] Interventions from the outside, such as acts of terrorism or natural disasters, are more likely to affect centralised WWTPs, whereas distributed on-site systems are expected to be more secure due to their scattered character. [19] It is also worth emphasizing that while failures of large-scale plants are usually detected without significant time lag, malfunctions of on-site systems without monitoring or alarm system can go on for weeks before they are detected and fixed. Decision makers, apart from other criteria, must consider the reliability and potential consequences of failure of each system.

Centralised plants benefit from economies of scale and as such are very cost-effective for the densely populated urban centres. Unfortunately, the past efforts to achieve maximum sewerage connection rate resulted in the need of construction of additional sewers, consequently increasing the length of sewer per user and per capita cost. [35]

Another economic consideration when choosing the optimal decentralization rate is the fact that large WWTPs are usually designed for a horizon of decades, therefore in the initial years of the WWTP operation, the available capacity is exploited only partly, resulting in increased unit costs. Moreover, construction of such projects requires considerable investments for a prospective uncertain scenario. Small decentralized systems, on the other hand, can be constructed with the 'pay as you go' approach, meaning the system can be designed for current requirement, and in case of insufficient capacity the system can be extended gradually. [11]

Due to the uncertainty of future weather conditions and increasing risk of more extreme weather conditions associated with climate change, in combination with expected global human population growth, the adaptive capacity of urban infrastructure systems has been considered recently as one of the sustainability indicators. This capacity is often referred to as "resilience" [54]. In general, decentralized systems are considered more resilient in cases of unprecedented external changes such as population growth or decreased wastewater production as they do not require planning decades ahead and can be easily extended.

However, most urban areas in central and western Europe already have existing sanitation infrastructure. In addition, centralised wastewater infrastructure had been considered the optimal

solution for decades, and introduction of a new approach is often hindered by institutional regulations established over long period of time. [66]

Until recently, simple cost benefit analysis (CBA) was utilised for selecting a wastewater treatment technology, only comparing initial and operational costs with the efficiency of pollutant removal. Nowadays, other factors apart from cost and efficiency shall be also taken in consideration [39]. Capodaglio has categorized these sustainability-influencing factors into three main groups: environmental, economic and social [11].

The environmental factors encompass protection of receiving environment by removing nutrients and organic matter, conservation of ecosystems and eliminating threats to human health by eradicating pathogens and micropollutants, opportunities for water reuse and nutrient recycling, energy requirements and possibilities of energy recovery. [11]

The economic factors are investment, operational and maintenance costs, costs per removed unit organic pollutant, costs of sludge disposal, and labour needed for the construction and operation. [11]

Social factors influencing the selection of most suitable technology are character of the settlement (urban/rural), associated nuisances such as odour or aesthetics problems, urban planning, regulations and legislation, public acceptance and operating fees. [11]

According to Jenssen et al., there is no universal approach or the most sustainable solution to urban water management. In every municipality, there are numerous factors that need to be taken in consideration, such as investment and operational cost, efficiency of treatment, recycling opportunities or social issues. [16]

It is now widely acknowledged that for most settlements, the optimal solution is neither fully decentralised nor fully centralised infrastructure. Consequently, a new approach to urban water management has developed, merging both centralised and decentralised systems. This so-called “hybrid form” of water management combines the advantages of centralised WWTP resulting from economics of scale for densely populated areas of given region with smaller on-site systems implemented where it would not be cost-effective anymore to extend the sewage network. [90] Financial, environmental, social and other aspects need to be taken in consideration during the process of estimating the optimal degree of centralisation (ODC), making it a complex and case-specific engineering task. [83]

The slow transition from centralised infrastructure to decentralised solutions brings forward many technological challenges and can have far-reaching socio-economic impacts. Comprehensive

evaluation of these impacts is a complex problem because multiple criteria need to be considered to achieve a variety of objectives, and usually the conclusions are only site-specific for a given case study. Various attempts at developing a universal methodology for determination of ODC have been presented.

Sitzenfrei et al. presented Virtual Infrastructure Benchmarking (VIBe), a tool for integrated city-scale analysis that uses computer generated stochastic data sets which eliminates the cumbersome work of data gathering, digitalisation, model building, calibration, and validation. The aim of the research paper was to determine the possible impact of implementation of decentralised infrastructure on existing sewage networks and treatment plants. It was determined that the impact of changes in water demands is higher for small systems (with population between 70 000 and 100 000). [83]

Poustie et al. used a multi-criteria decision analysis (MCDA) to evaluate the technical, economic, environmental and resilience performance of water supply, wastewater and stormwater systems. Five experts were asked to assign a score to multiple alternatives and varying degrees of centralisation. The analysis of wastewater systems showed that with decreasing centralisation, technical performance declines, while economic, environmental and resilience performance all improve. For stormwater systems, economic performance increased with centralisation, whereas technical and environmental performance declined with centralisation. Resilience performance showed only slight decreasing trend with centralisation. [70]

Eggiman et al. developed a planning tool for sustainable network infrastructure planning (SNIP). The model evaluates the most expedient wastewater management configuration and finds the optimal layout of treatment plants, sewers, and pumps, by taking into account several design parameters, such as minimum and maximum trench depth, minimum slope, wastewater production, diameter, etc., and cost parameters, such as operation cost of the sewers, pumps and WWTPs, lifespan of pipes and WWTPs, electricity, etc. The model confirmed the expectation that the more complex terrain and the lower population density, the lower the ODC. On a real case study it was determined that the ODC was lower than current level. [25]

Jenssen et al. compared two wastewater alternatives in terms of sustainability and pointed out that sustainability assessments should always consider local, regional, and global scale. The article sums up current large-scale technologies of nutrient removal in Norway and compares the centralised scenario to a source separating recycling system in terms of energy requirements, emissions, reuse options and water consumption. [39]

Capodaglio assembled a set of factors that influence sustainability of a wastewater treatment system, and expressed the view that there is no universal solution to sustainability evaluation, because sustainability indicators that are successfully applied to one case may not be applicable to another, due to different expectations, requirements, local conditions and definitions of "sustainability". According to Capodaglio, the selected set of indicators should always reflect geographic and demographic characteristics of each case for optimal results. [11]

2.5 Related legislation

For EU members, the most substantial piece of legislation concerning water protection is the **Council Directive 2000/60/EC** of the European Parliament and of the Council establishing a framework for the Community action in the field of water policy, or Water Framework Directive (WFD). The WFD aims at maintaining and improving quality of the aquatic environment by setting environmental objectives for surface water, groundwater, and protected areas. Member states are obliged to prevent deterioration of the status of all water bodies and to limit or prevent input of pollutants to groundwater by implemented the necessary measures.

Council Directive 91/271/EEC on Urban Waste Water Treatment (UWWTD) aims to eliminate the adverse impacts on waterbodies from discharges of urban wastewater. In 1998, Directive 98/15/EC amending Directive 91/271/EEC was issued to define requirements for discharges to sensitive areas which are prone to eutrophication. It is worth mentioning that all surface waters in Czech Republic are defined as sensitive areas, hence more stringent regulations apply to them.

As mentioned above, this Directive set staged deadlines for implementation of wastewater collecting systems, secondary treatment, and more stringent treatment facilities in sensitive areas. The deadlines were due to compliance by the end of 1998, 2000, and 2005, depending on the population equivalent and type of receiving waters. For Czech Republic, due to latter accession to EU, transition period has been negotiated until the end of 2010.

To meet the objective of achieving good ecological status of waterbodies as stated in WFD, the UWWTD obliges the member states to ensure appropriate treatment of discharges from agglomerations of less than 2000 PE, before they can be discharged to fresh-water and estuaries. However, the UWWTD does not state any duty in relation to these agglomerations.

The agricultural application of sewage sludge is governed by **Council Directive 86/278/EEC**, on the protection of the environment, and in particular of the soil, when sewage sludge is used in agriculture. This Directive restricts agricultural application of sewage sludge to prevent adverse

impacts on the environment and human health and to maintain the quality of the soil, surface waters, and groundwater, and sets maximum allowed concentrations of seven heavy metals that are possibly toxic to plants and humans

In Czech Republic, the WFD and the UWWTD have been implemented into the **Water Act 254/2001 Col.**, as amended. The Water Act provides the conditions for the use of surface waters and groundwater, integrates the issue of flood protection, establishes the role and competences of public authorities, and defines the obligations of natural and legal persons in relation to water protection. The Water Act lays down the definition of wastewater and defines the obligations arising from discharging wastewater into the environment.

Major piece of national legislation regarding wastewater treatment is the **Act 274/2001 Col**., on public water mains and sewerage, as amended. This act incorporates requirements of the UWWTD to regulate development, construction and operation of water supply and sewerage systems serving public, connections to them, as well as the competence of the bodies of territorial self-governing units and administrative authorities in this section.

Government Regulation No. 401/2015 Col., on the indicators and values of permissible pollution of surface water and wastewater treatment, details of the permit to discharge wastewater into surface water and sewerage systems and sensitive areas, establishes threshold values for all major pollutants and indicators of pollution for surface waters and wastewaters and lays down the requirements for obtaining a licence to discharge wastewater to surface waters, sewerage, and sensitive areas, in accordance with the EU regulations.

The **Act No. 185/2001 Col.**, on waste and the amendment of some other acts, defines sewage sludge from WWTPs and septic tanks as waste. The Act restricts sludge use in agriculture and defines conditions under which the use of sludge is forbidden.

Decree 501/2006 Col., on general land use requirements, defines priorities in stormwater management on building land. According to this Decree, preferred way of stormwater management for new houses is infiltration. The Decree defines the minimum ratios of the ground size suitable for infiltration to the total ground size for detached family houses, terraced houses and blocks of flats. If infiltration is not possible, stormwater must be retained on the site, then gradually discharged into separate stormwater network, and transported to receiving waterbody. If this is not possible, it is allowed to discharge stormwaters into combined sewage network.

2.6 Authorisation of wastewater discharges in the Czech Republic

The authorisation process of both small-scale and municipal WWTPs in Czech Republic is governed by Water Act 254/2001 Col. This act states that wastewater discharges to the environment should follow maximum permissible quantity of wastewater and its pollution based on a permit issued by water authority. However, discharges from WWTPs with designed capacity lower than 50 PE may not be subject to this obligation, as long as essential part of the treatment is system is a product bearing CE marking.

A further important administrative difference lies in the fact that while large municipal WWTPs need a *building permit* for construction, for small-scale WWTPs for less than 50 PE, in some cases *notification* may be sufficient, according to Building Act No. 183/2006 Col.

In addition, the system of controlling the effluent quality is different. WWTPs that were authorised in a water authority proceeding are subject to § 38 of the Water Act, i.e. the operator of the WWTP is obliged to periodically report to the water authority on volume and quality of the discharged effluent. The frequency of sampling depends on the WWTP size category and must be in accordance with Government Regulation No. 401/2015 Col. [67]

WWTPs bearing the CE marking that were authorised on the basis of notification do not have the obligation of regular reporting. Instead, the owner of the WWTPs is required to carry out technical checks once every two years by inviting person authorised by Czech Ministry of Environment. The owner reports the results of the check to relevant water authority, and if the WWTPs fails to provide effluent of sufficient quality, the owner is obliged to ensure its proper functioning within 60 days. [67]

The differences between the two approaches are summarized in *Table 1*.

Table 1. Comparison of different approaches to WWTP authorisation in Czech Republic

approach	WATER AUTHORITY PROCEEDING	PRODUCT APPROACH
scale	large-scale/municipal WWTPs or small-scale WWTPs in vulnerable localities	WWTPs < 50 PE bearing the CE marking
construction authorisation	building permit	notification
control of operational aspects	periodical sampling and reporting to water authority	periodical checks performed by person authorised by Ministry of Environment
frequency of controls	based on the WWTP size, for < 500 PE minimum 4 times a year	once in every two years (no samples taken)
wastewater discharge permit validity	maximum 10 years	indefinite

2.6.1 Water authority proceeding – authorisation of municipal WWTPs

Before a large-scale centralised WWTP can be constructed and operated, participation in a *water authority proceeding* must take place. Small-scale WWTPs for less than 50 PE must take part in such proceeding only if there are special local conditions or circumstances that prevent the water authority from authorising small-scale WWTPs on the basis of notification.

If the water authority approves the application, water management permit is issued. If the application concerns water management which can be only carried out when using special structure, water management permit and building permit must be issued at the same time. Therefore, approved applications proposing a construction of a new municipal WWTP result in issue of both water management permit and building permit, linked by mutual conditionality.

According to the Water Act, discharges of wastewater into surface water or groundwater are permitted only with a permission issued by competent water authority. The permission for the management of surface or groundwater, also referred to as *water management permit*, shall specify the purpose, scope, obligations and, if applicable, the conditions under which such authorization is granted. Permissions are issued for a limited time. For wastewater discharges, the period of validity cannot exceed 10 years, for extremely dangerous substances it is limited to 4 years. [1]

The water management permit can either establish minimum required treatment efficiency for individual parameters such as chemical oxygen demand (COD_{Cr}), biological oxygen demand (BOD_5), and total suspended solids (TSS), and for some sites also for $N\text{-}NH_4^+$, total nitrogen (TN), and total phosphorus (TP), or set emission limits for each parameter. Water authority sets these limits for each site using combined approach which takes into account receiving waterbody status, environmental quality standards, water use requirements, and target environmental condition of surface waters. This approach considers both values of permissible surface water pollution (environmental quality standards, EQS) and acceptable values of wastewater pollution (emission standards), and takes into account best available technologies (BATs). Emission limits are set with regard to all these factors up to the maximum value equal to the relevant emission standard. If the calculated emission levels could not be reached even after implementation of BATs, the emission level will be set as the BAT associated emission level (BAT-AEL).

Emission standards and BAT-AELs for individual parameters are set out in the Government Regulation No. 401/2015 Col. Emission standards and BAT-AELs are expressed as minimum treatment efficiencies, acceptable values (p), maximum values (m) and average values, and they are divided in five categories according to the size of agglomeration. In addition, this piece of

legislation sets out the minimum annual sampling frequency, required type of sample and requirements for minimum efficiency.

Acceptable values (p) are set for COD_{Cr}, BOD_5, and TSS. These values can be exceeded to some extent. The number of acceptable non-compliant samples depends on the total sum of samples. Average values are set for $N\text{-}NH_4^+$, total nitrogen (TN), and total phosphorus (TP), and cannot be exceeded. Maximum values (m) are set for COD_{Cr}, BOD_5, TSS, $N\text{-}NH_4^+$, TN, and TP, and cannot be exceeded.

For municipal WWTP treating wastewater from agglomerations of less than 500 PE, the minimum number of samples per annum is 4. The samples are a two-hour mixed sample obtained by combining 8 increments of the same volume taken over 15 minutes. When there are 4-7 samples in total per annum, one sample can exceed the 'p' value. However, the non-compliant sample still must be in the range from 'p' to 'm' value.

For the size category of municipal WWTPs treating wastewater from agglomeration of less than 500 PE, Government Regulation No. 401/2015 does not set any emission standards for $N\text{-}NH_4^+$, TN, and TP. However, water management permit can introduce emissions limits also for these parameters.

*Table 2 r*eviews the acceptable values 'p', maximum values 'm', minimum required efficiencies for individual parameters, and shows achievable concentrations and efficiencies when using BATs.

Table 2. Emission standards and required efficiencies for municipal WWTPs for agglomerations of less than 500 p.e [67]

WWTP < 500 PE		COD_{Cr}		BOD_5		TSS	
		p	m	p	m	p	m
Required concentration	[mg/l]	150	220	40	80	50	80
Achievable conc. (BAT-AEL)	[mg/l]	110	170	30	50	40	60
Minimum required efficiency	[%]	70		80		-	
Achievable efficiency (BAT-AEL)	[%]	75		85		-	

2.6.2 Product approach – WWTPs for less than 50 PE bearing the CE marking

According to the Act No. 183/2006 Col., on town and country planning and building code (Building Act), WWTPs can be authorised either based on *building permit*, or *notification*.

Building permit is necessary for construction of WWTPs with designed capacity over 50 PE [67]. To obtain the building permit, statements of all concerned parties are necessary, and the process of building permit authorisation can be complicated and lengthy. If domestic WWTPs had to be authorised with the same process as large municipal WWTPs, it would impose a significant burden on the regional authorities in terms of funding and personnel. Moreover, it could represent a deterring factor for people considering acquiring a domestic WWTP.

Therefore, new *product approach* was introduced which is less cumbersome to administer, while allowing sufficient control over discharges of wastewater and preventing pollution of the environment. The idea behind this approach is that the product will be tested according to a harmonised European standard to ensure balanced and fair results. If the product is capable of producing effluent of quality compliant with given requirements, in this case with minimum treatment efficiency requirements which differ for each WWTP category, it is granted the CE marking.

Using this approach, the administrative process of authorizing small-scale WWTPs is less complicated, as it can be assumed that products that have been granted the CE marking are reliable enough to produce effluent of sufficient quality to meet requirements stated in Government Regulation No. 401/2015 Col. Therefore, under standard circumstances and in locations that do not have any special restrictions or site-specific conditions, no building permit is required. Instead, notification is submitted to the responsible authority.

To address higher environmental vulnerability of some areas, three categories of the CE marking for small-scale WWTPs have been introduced. Government Regulation No. 401/2015 describes criteria for inclusion into one of three categories:

Category I encompasses domestic WWTPs intended for discharges into surface waters where it can be demostrated that environmental quality standards (EQS) will not be exceeded.

Category II has higher requirements of carbonaceous pollution removal and requires nitrification to protect the receiving waterbody, especially for waterbodies sensitive to ammonia pollution, and for rivers and streams where EQS and water use requirements may not be achieved due to low flows. Domestic WWTPs in category II must provide longer sludge retention times as compared to category I, e.g. greater volume of aeration tank or another structural element that will ensure higher concentration of biomass.

Products included in *category III* are required to ensure greater nitrification, to partly remove nitrogen by denitrification and to eliminate phosphorus as these types of products discharge the effluent to waters with more stringent protection requirements, e.g. raw water sources for water supply schemes etc.

Table 3. Minimum required efficiency for domestic WWTPs expressed as percentage of removed pollution [68]

Category		COD_{Cr}	BOD_5	$N\text{-}NH_4^+$	TN	TP
I	[%]	70	80	-	-	-
II	[%]	75	85	75	-	-
III	[%]	75	85	80	50	80

CE marking is also required for discharges to groundwater. The criteria for certifying a WWTP as suitable for groundwater discharges are listed in Government Regulation No. 57/2016. Col. The requirements for obtaining the CE marking are set as required efficiencies of treatment.

Table 4. Minimum required efficiency for WWTPs discharges into groundwater [68]

Discharges to groundwater	COD_{Cr}	BOD_5	TN	TP
efficiency [%]	90	95	50	40

Government Regulation No. 57/2016. Col. also sets emission standards for discharges to groundwater, only as the "m" values. The following table summarizes the required standards for houses and buildings for recreation:

Table 5. Emission standards for discharges into groundwater from houses and buildings for recreation [111]

Size category (PE)		value "m" [mg/l]				
		COD_{Cr}	BOD_5	$N\text{-}NH_4^+$	TSS	TN
< 10	[mg/l]	150	40	20	30	-
10 – 50	[mg/l]	150	40	-	30	30
> 50	[mg/l]	130	30	-	30	20

For wastewaters discharged from tourist accommodation sites, required standards are as follows:

Table 6. Emission standards for discharges into groundwater from tourist accommodation sites [111]

value "m" [mg/l]				
COD_{Cr}	BOD_5	TSS	TP	TN
130	30	30	8	20

In addition, Government Regulation No. 57/2016. Col. sets emission standards for microbiological pollution. The value "m" for Escherichia coli and enterococci is 150 and 100 CFU/100 mL, respectively.

3 THESIS GOALS

This thesis aims to compare centralised and decentralised wastewater and stormwater management in three selected municipalities in the Czech Republic based on environmental, energy and operation related criteria.

Czech legislation imposes differing requirements on effluent quality for centralised WWTPs of less than 500 PE and for small domestic WWTPs. The requirements for WWTPs of less than 500 PE are more stringent as compared to small on-site systems. Therefore, more polluting substances can enter the waterbody when domestic WWTPs are used. In the first part of environmental assessment, balance of pollutants discharged into receiving environment will be calculated for each municipality based on three different scenarios – strictest and tolerant scenario will be based on effluent quality requirements set by Government Regulation No. 401/2015 Col, and declared efficiency scenario will compare normally achieved effluent concentrations of both centralised and decentralised systems.

Second aim of this thesis is to calculate energy requirements for centralised and decentralised wastewater management and compare them.

The thesis will also consider feasibility and possible environmental benefits arising from reclamation of the treated wastewater or stormwater and thus reducing the water and energy consumption. Energy consumption of stormwater and treated wastewater reuse systems will be calculated and compared to energy consumed to treat the same volume of potable water.

The last part of the thesis will compare operational reliability of centralised and decentralised treatment options based on informed opinions and literature research, and calculate amount of pollution that can enter the receiving environment when possibility of failure is taken into account.

4 CASE STUDIES

This thesis follows up on thesis 'Analysis of optimal degree of decentralisation of small municipalities drainage and treatment systems' which was defended at the CTU Faculty of Civil Engineering in 2016, and which compared the centralised and decentralised options from the economic point of view. [33]

The municipalities that were subject to aforementioned thesis will now be investigated in terms of environmental, energy and operation-related aspects. Quick introduction of the municipalities follows.

4.1 Tisovec

Tisovec is a municipality in the Pardubice region, in the Chrudim district, with a total population of 328 inhabitants. [18] It comprises five local parts - Tisovec, Dřeveš, Kvasín, Otáňka, and Vrbětice, with distances between each other approx. 1 kilometre.

River Ležák flows through the local part Tisovec. Its right-hand tributaries, Havlovický and Dřevešský stream, run across the local parts Kvasín and Dřeveš, respectively. Local parts Otáňka and Vrbětice are not intersected by any watercourse.

All local parts are connected to public water supply. There is no sewage network, some parts of the municipality are drained by stormwater drainage discharging into river Ležák. Major part of the population uses waterproof cesspits to accumulate wastewater, which is periodically transported to nearest WWTP. About one third of population uses septic tanks discharging into stormwater drainage which is a practice non-compliant with the legislative regulations.

According to Zoning Plan of Tisovec from 2009, wastewater from Tisovec shall be transported by separate sewage network to the WWTP in Včelákov 3 kilometres away. However, the capacity of the existing WWTP would have to be extended to allow the connection of Tisovec. [18] The Zoning Plan also mentions available land in Dřeveš that could be used for construction of a new WWTP that would treat wastewater from all the local parts of Tisovec.

The Development Plan of Water Supply and Sewage Systems for Pardubice Region (PRVKÚK) proposes continuation of the status quo, i.e. decentralised wastewater management in septic tanks and cesspits, with the exception of local part Dřeveš. The unauthorized discharges from septic tanks are to be eliminated, and the septic tanks shall be retrofitted with soil filters to comply with the

Water Act 254/2001 Col. The Development Plan also points out the necessity to revise the technical condition of existing cesspits.

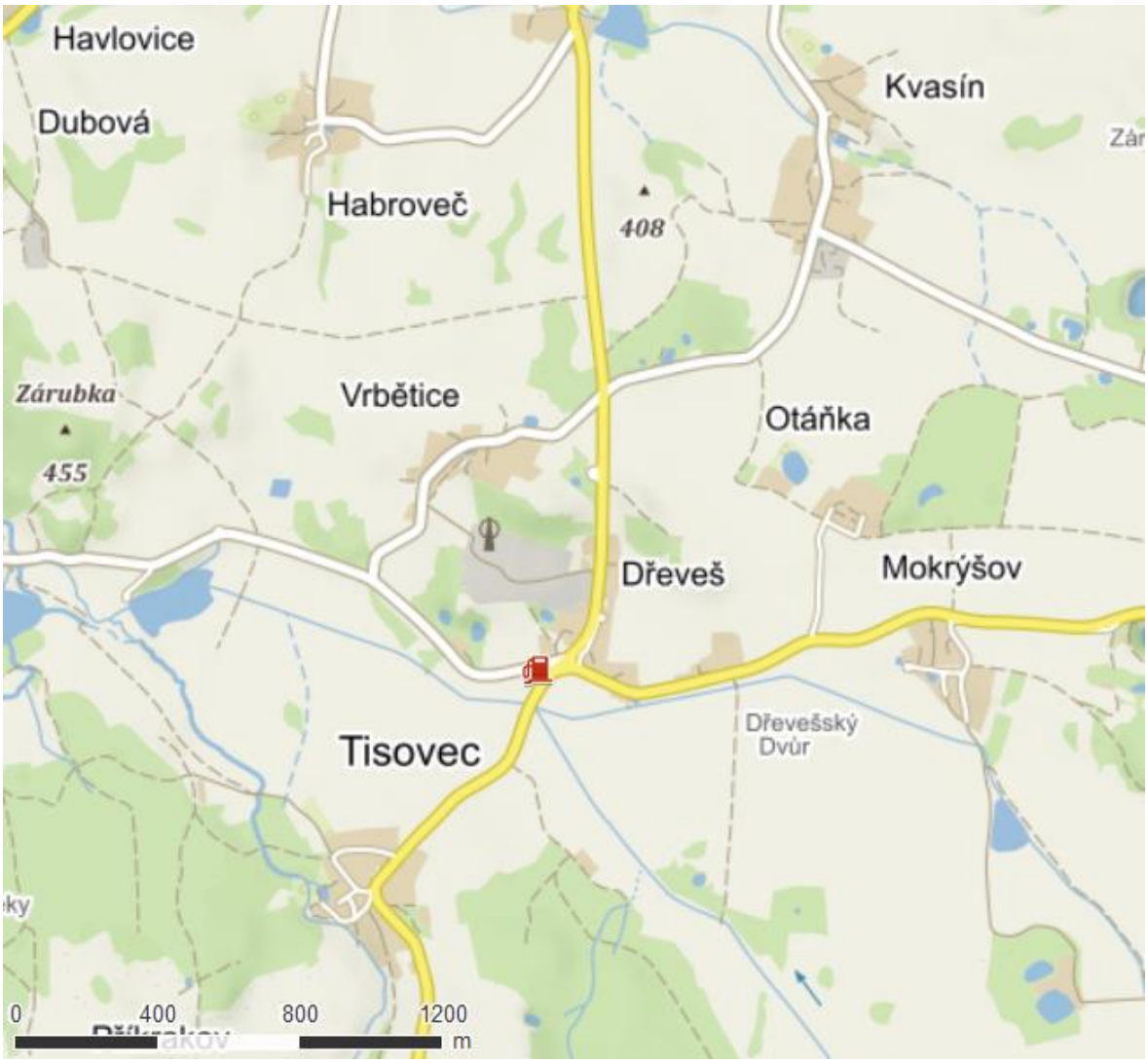

Figure 14. Tisovec, Dřeveš, Kvasín, Otáňka, and Vrbětice [112]

In the local part Dřeveš, the Development Plan suggests revision of existing stormwater drainage for possible use as combined sewage network, and construction of a new WWTP with designed capacity 120 PE (15 m^3/day). With regard to the small size of the agglomeration, suggested technological solutions are either a stabilisation pond, or constructed wetland. The assumed realisation date is after 2015.

4.2 Chlumětín

Chlumětín is a municipality in the Vysočina region, in the Žďár nad Sázavou district, with a population of 203 [18]. It has only one local part of the same name stretched along road going

through the municipality, and two small settlements, Krejcar and Paseky. The municipality is intersected by the Chlumětínský stream, which then flows into river Chrudimka.

The municipality lies in protected countryside area (CHKO) Žďárské vrchy. The territory of the municipality contains one nature reserve (PR) Volákův kopec. Transition mires and quaking bogs Chotáry approx. 2 kilometres west from Chlumětín are defined as Site of Community Importance (SCI, in Czech EVL) under the Natura 2000 network.

Figure 15. Chlumětín, Krejcar and Paseky [112]

Drinking water is delivered by a public water supply. Majority of the population (75%) uses septic tanks connected to a combined sewage network built in 1974, which is still in satisfying condition. Wastewater is transported to three stabilisation ponds, treated wastewater is discharged in the Chlumětínský stream. The rest of inhabitants uses cesspits. The accumulated wastewater from cesspits is transported by trucks to WWTP Hlinsko approx. 10 km away from Chlumětín.

The Development Plan of Water Supply and Sewage Systems for Vysočina Region proposes completion of the combined sewage network and construction of a new WWTP. In 2014, spatial planning decision for the WWTP was issued, but to this date the construction has not begun. It describes the WWTP as series of aerated stabilisation ponds, with an aerated biological reservoir, nitrification tank, and sedimentation tank.

The Plan expects realisation by 2030. In case the WWTP construction will not be realised, the Plan suggests decentralised management – in this case, wastewaters shall be disposed of individually, e.g. in domestic WWTPs or cesspits.

4.3 Bořice

Bořice is a municipality in the Pardubice region, in the Chrudim district, with a population of 164. [18] It is divided into two local parts, Bořice and Podbor.

There are no natural waterbodies in the area. South of the municipality, there is a beginning of a melioration ditch that discharges into the river Novohradka.

There is no public water supply in the village, water is supplied individually by local wells. The municipality plans connection to the public water supply when there is enough funding.

As of now, there is no sewage network in the municipality. Wastewater from majority of inhabitants (79%) is stored in cesspits and exported to the nearest WWTP Hrochův Týnec, the rest of population uses septic tanks. [55]

In 2016, the municipality applied for a zoning decision for construction of a separated sewage network. The network will transport wastewaters from both Bořice and Podbor to existing WWTP in Hrochův Týnec. The wastewater will be conveyed by gravity and where necessary it will be pumped. The design includes two pumping stations. According to the building permit, the construction of the sewage network is made conditional upon the increase of capacity of WWTP Hrochův Týnec from 2300 PE to 2500 PE. However, according to PRVKÚK, the WWTP was designed with enough capacity to allow connection of Bořice without modifications to existing plant.

In May 2017, total sum of 4 000 000 CZK (approx. 160 000 EUR) was allocated by the Pardubice region for construction of separated sewage network in the local part Podbor. [72]

Stormwater is transported by shallow stormwater drainage. According to the Development Plan of Water Supply and Sewage Systems, some septic tank overflows discharge wastewater into the stormwater drainage.

Figure 16. Map of Bořice and Podbor [112]

5 METHODS

5.1 Environmental assessment

As already stated above, the authorisation process, establishment of effluent quality requirements, and reporting on operational performance are different for municipal and small-scale WWTPs. For municipal WWTP, water authority lays down emission limits ranging from emission standards (the least stringent value) to BAT-AELs (the most stringent value). For WWTPs with capacity less than 50 PE, treatment requirements are expressed as minimum efficiency and they are differentiated according to the category of WWTP.

For the investigated centralised options, the exact values of emission limits for municipal WWTP are not known. Similarly, for decentralised solutions, required efficiencies would vary depending on the small-scale WWTP category. Therefore, multiple scenarios will be compared:

I. **Strictest scenario** assumes the emission limits imposed by water authority will be the most stringent ones, i.e. for municipal WWTP the set limits will equate to BAT-AELs, and for small-scale WWTPs criteria for category III will be used.

II. **Tolerant scenario** expects the limits to be set as the maximum allowable value, i.e. emission standards for municipal WWTPs, and minimum efficiency requirements for small-scale WWTPs in category I.

III. **Declared efficiency scenario** will compare average treatment efficiencies for proposed centralised solution in every municipality, i.e. constructed wetland (Tisovec), series of biological aerated ponds (Chlumětín), and municipal WWTP with designed capacity 2300 PE (Bořice) to average and guaranteed efficiencies achieved by small-scale WWTPs.

The emission limits for municipal WWTPs are set with regard to normal fluctuations in effluent quality caused inter alia by varying characteristics of inflowing wastewater, i.e. its dilution due to rainfall events or temporal presence of substances that are toxic to microorganisms in aeration tanks and affect the final effluent quality. In other words, the requirements are laid down with a certain tolerance, but effluent quality that can be achieved under normal operating conditions (dry weather flow, absence of substances that have inhibitive effect on biological organisms) can be much higher.

Overall treatment performance of small-scale WWTPs is estimated based on data from a research conducted in a Czech public research institution T. G. Masaryk Water Research Institute (TGM WRI)

that tested the performance of 24 different systems for treatment of domestic wastewater. The systems were subdivided into 4 technological categories: sequencing batch reactor (SBR), aerated bioreactor (AB), aerated bioreactor with advanced phosphorus removal (AB + APR), and membrane bioreactor (MBR). [38] The results of the 38 weeks long testing are summarised in *Table 7*.

Table 7. Overall treatment performance of small-scale WWTPs [38]

system		SBR	AB	AB + APR	MBR
no. of tested systems		5	14	3	2
BOD_5	%	98	98	99	100
COD_{Cr}	%	93	91	95	96
TSS	%	96	93	98	100
$N\text{-}NH_4$	%	94	88	77	97
TP	%	69	39	93	57

Some of the producers of treatment systems for domestic wastewater provide guarantees of effluent quality, either as “p” and “m” values, or expressed as efficiency. For the purpose of this thesis, one major producer was selected for calculating daily pollution loading based on these guaranteed values, and compared to pollution produced when considering overall treatment efficiencies. The “p” and “m” guaranteed values for individual technological option are summarised in Table *8.*

Table 8. ASIO. Small-scale WWTPs - guaranteed effluent concentrations (AB - ASIO-VARIOcomp; MBR - ASIO-VARIOcomp ULTRA; AB + SF - ASIO-VARIOcomp + vertical soil filter AS-ZEON; ST + SF – septic tank AS-ANASEP + vertical soil filter AS-ZEON)

ASIO guaranteed values	AB		MBR		AB + SF		ST + SF	
	p	m	p	m	p	m	p	m
	[mg/l]		[mg/l]		[mg/l]		[mg/l]	
BOD_5	25	40	5	10	5	10	25	40
COD_{Cr}	90	150	70	90	40	70	90	150
TSS	30	40	3	6	10	20	10	20
$N\text{-}NH_4$	15	20	15	20	10	20	10	20
TP	8	10	8	10	6	8	8	10

The compared products are: **ASIO-VARIOcomp**, wastewater treatment system consisting of primary sedimentation tank, aerated bioreactor with biomass carrier, and secondary sedimentation tank. Sludge from the last tank can be pumped back into primary sedimentation tank. Effluent from secondary sedimentation tank is pumped by air lift pump to the outlet. **AS-VARIOcomp K ULTRA** has similar construction as the basic model, but the wastewater is filtered through a membrane

system located in the bioreactor. The high-quality effluent can then be reused for irrigation, flushing, etc. **AS-ANASEP** is a four-chamber septic tank with baffles which must be combined with a secondary treatment step, such as vertical soil filter **AS-ZEON**.

For all scenarios, following assumptions have been made:

1) an average Czech person generates 90 litres of wastewater daily [89]

2) the pollution loading per capita per day is 60, 120, 55, 11, 7, and 2.5 g/p.e/day for BOD_5, COD_{Cr}, TSS, TN, N-NH_4, and TP, respectively

3) the amount of infiltration/inflow (I/I) will be taken as 15% of dry weather flow (DWF) for municipalities Tisovec and Bořice, where the envisaged sewage network would be newly built and therefore in better condition than in Chlumětín, where the centralised option proposes use of an existing sewage network. For Chlumětín, the estimated I/I is 25%.

4) effluent concentration based on values "p" and "m" was calculated as an average value from 4 samples (minimum number of samples for municipal WWTPs for < 500 PE), of which 3 samples are equal to the "p" value, and one sample corresponds to the "m" value.

Daily generation of pollution in each municipality is summarised in *Table 9.*

Table 9. BOD_5, COD_{Cr}, TSS, TN, N-NH_4, and TP daily production in each municipality

municipality		Tisovec	Chlumětín	Bořice
population equivalent		363	203	161
parameter	[g/PE/day]	[kg/day]	[kg/day]	[kg/day]
BOD_5	60	21.78	12.18	9.66
COD_{Cr}	120	43.56	24.36	19.32
TSS	55	19.97	11.17	8.86
TN	11	3.99	2.23	1.77
N-NH_4	7	2.54	1.42	1.13
TP	2.5	0.91	0.51	0.40

5.2 Energy assessment

Energy assessment is divided into two parts:

a) energy requirements for wastewater treatment – centralised WWTP compared to small-scale systems

This part compares energy consumption of a municipal WWTP to energy requirements of series of decentralised systems.

The compared decentralised systems for wastewater treatment are: **ASIO-VARIOcomp** (aerated bioreactor), **AS-VARIOcomp K ULTRA** (membrane bioreactor), both described in previous section, and **AS-IDEAL PZV** which is an SBR reactor using alternating phases of aeration and sedimentation. Septic tank **AS-ANASEP** does not have any energy requirements and therefore will be not included in the energy consumption assessment.

All three decentralised systems come in multiple size variants which have different energy requirements. For the purpose of the comparison it was assumed that each domestic WWTP will be used to its full capacity.

According to Ing. Karel Plotěný from company ASIO that produces compared small-scale WWTPs, the daily operation time depends on the technology used. ABs consume energy for aeration all day, while MBRs often have redundant oxygen supply and thus it is possible to introduce short cycles without aeration. On average the MBRs operate 21 hours per day [68]. SBRs do not consume energy during the sedimentation phase which takes about one hour per day.

Table 10. Size variants of SBR, MBR and AB systems and their respective input power

SBR		MBR		AB	
operation time 23 h/d		operation time 21 h/d		operation time 24 h/d	
EO	[W]	EO	[W]	EO	[W]
2-3	90	3-5	150	3-7	40
4-5	90	6-10	170	6-10	50
6-7	90	11-17	390	10-13	95
8-10	110	18-24	400	13-17	75
-	-	-	-	18-25	95

b) on-site reuse systems – energy requirements for operation compared to energy that would be needed for treatment and distribution of the same volume of potable water

This part calculates energy requirements for pumping of accumulated stormwater/treated wastewater, and compares them to energy needed for to treat and distribute corresponding volume of potable water.

Two options of on-site reuse will be investigated:

- stormwater accumulation only – reuse only for irrigation
- stormwater and treated wastewater accumulation – reuse for irrigation, flushing, and laundry washing

STORMWATER ACCUMULATION ONLY

The volume of stormwater that is available for collection and reuse can be calculated as:

$$V_{available} = \frac{h}{1000} * A * \psi * \eta, \quad (5.1)$$

where h is the long-term normal [mm], A is collecting area [m^2], ψ is surface runoff coefficient that depends on slope and material of roofing, and η is coefficient of filter loss [85].

The long-term precipitation normal 1981-2010 during the vegetation period from April to September for Pardubice region is 463 mm [16]. The average collecting area for a detached house is usually around 120 m^2. For steeply pitched roofs with hard surface the coefficient ψ equals to 0.8 [85]. Coefficient η was taken as 0.9 [85]. During heavy rainfalls, if the capacity of the tank will be reached, part of stormwater will overflow. This water is assumed to be 25% of total volume. The volume of stormwater available for collection was thus calculated as 30 m^3.

Volume of water required for irrigation is calculated as:

$$V_{required} = A_{irri} * P * n - V_{rain}, \quad (5.2)$$

Where A_{irri} is the irrigated area [m^2], P is specific irrigation requirement for lawn/crops [l/m^2/week], n is number of weeks when the garden will be irrigated, and V_{rain} is the volume that crops receive from natural precipitation, calculated based on precipitation normal during the vegetation period and garden area [85]. Average garden area was taken as 500 m^2, specific irrigation requirement for lawn as 30 l/m^2/week, and number of weeks as 27 (vegetation period from April to September = 214 days). $V_{required}$ was 173.5 m^3 which is substantially higher than what is available for accumulation. The volume of the accumulation tank with 28-day storage was thus calculated as:

$$V_{accum} = V_{available} * \phi = 30 * \frac{28}{214} = 3.93\ m^3 \quad (5.3)$$

The tank was designed for 4 m^3. Selected system for stormwater reuse is **AS-RAINMASTER** which is a unit that filters and pumps the water from accumulation tank to irrigation devices, monitors the level of accumulated stormwater and in case of running out of stormwater starts pumping potable water from the distribution network. The producer states that the unit is suitable for stormwater and greywater reuse. Selected type was AS-RAINMASTER ECO 10 with maximum flow 10 l/min and required input power 0.09 kW [3]. Assumed daily operating time was calculated as:

$$T = \frac{4000\ l}{10\ l/min} = 0.24\ hours/day \quad (5.4)$$

Energy requirements for a water treatment plant (WTP) and distribution network are dependent on the raw water quality, used technologies and processes, age of mechanical equipment, size of

the plant and served area, storage capacity, topographical conditions, etc. According to data of company Veolia Czech Republic which provides water supply to approximately one third of Czech population, the average power input needed to treat and supply water for one person was 33 kWh in 2015 [97], including losses during distribution. To produce 1,000 cubic meters, 523 kWh is needed on average [97]. Comparison of those two numbers shows that the WTP has to produce approximately 170 l/day to provide water supply to one person when losses in the distribution network are taken into consideration. Value of 523 kWh/m^3 roughly corresponds to energy requirement estimated by US EPA - 1500 kWh/million gallons for surface water and 1800 kWh/million gallons for groundwater [96], which equals to 0.4 and 0.48 kWh/m^3.

STORMWATER AND TREATED WASTEWATER ACCUMULATION

Potable water that could be replaced with water of non-potable quality was calculated based on following assumptions:

- Average household in Western Europe runs 173 washing cycles per annum, and each washing cycle needs approximately 60 litres of water [65]. Daily average amount of water consumed for washing by every household is thus 28.4 l.
- Low-flush toilets use 4.5 litres per flush or even less [31]. Given that the average person flushes five times a day [50], the daily water consumption for flushing is 22.5 l.
- The amount of water needed for irrigation in the vegetation period is dependent on the irrigated area, type of crops, and season. Calculated $V_{required}$ in previous section was 173.5 m^3 for vegetation season, which equals to 0.81 m^3/household/day.

Potable water that could be replaced with reclaimed stormwater and treated wastewater is summarised in *Table 11*. The table shows daily production of treated effluent (estimated as 90 l/PE/day as the removed solid part can be assumed to be negligible) and average amount of stormwater that is available per day. Water that is available to be reused is shown in the last column of the table.

Table 11. Potable water used in each household for laundry washing, toilet flushing, and irrigation

municipality		TISOVEC	CHLUMĚTÍN	BOŘICE
PE		343*	203	161
washing laundry	[l/d]]	28.4	28.4	28.4
flushing	[l/d]	74.93	52.50	41.16
irrigation	[l/d]	811	811	811
consumption (April - October)	[l/d]	914	892	880
consumption (November - March)	[l/d]	103	81	70

**only 343 PE are considered in Tisovec because pollution equivalent to 20 PE originates from a farm*

The dimensions of the tank for stormwater and treated effluent accumulation were calculated as follows:

Table 12. Dimensions of combined tank for stormwater and treated wastewater accumulation, vegetation period

	municipality		TISOVEC	CHLUMĚTÍN	BOŘICE
	persons/household		3.33	2.33	1.83
INFLOW	production wastewater	[l/d]	300	210	165
		[m^3/vegetation period]	64.1	44.9	35.2
	precipitation	[m^3/vegetation period]	40.0	40.0	40.0
	total	[m^3/vegetation period]	104.1	84.9	75.2
OUTFLOW	household requirement	[l/d]	103.3	80.9	69.6
		[m^3/vegetation period]	22.1	17.3	14.9
	irrigation	[m^3/vegetation period]	173.5	173.5	173.5
	total	[m^3/vegetation period]	195.6	190.8	188.4
	volume of the tank		13.6	11.1	9.8

Table 13. Dimensions of combined tank for stormwater and treated wastewater accumulation, non-vegetation period

	municipality		TISOVEC	CHLUMĚTÍN	BOŘICE
	persons/household		3.33	2.33	1.83
INFLOW	production wastewater	[l/d]	300	210	165
		[m^3/non-veget. period]	45.3	31.7	24.9
	precipitation	[m^3/non-veget. period]	20.6	20.6	20.6
	total	[m^3/non-veget. period]	65.8	52.3	45.4
OUTFLOW	household requirement	[l/d]	103.3	80.9	69.6
		[m^3/non-veget. period]	15.6	12.2	10.5
	irrigation	[m^3/non-veget. period]	0	0	0
	total	[m^3/non-veget. period]	15.6	12.2	10.5
	volume of the tank	[m^3]	2.9	2.3	1.9

The calculated dimension show that for non-vegetation period, smaller tank would be sufficient to cover the requirements for flushing and washing. Nevertheless, a small tank would not be able to provide sufficient accumulation of stormwaters in the vegetation period, and large portion of stormwater during heavy rains would have to be discharged due to insufficient capacity. Therefore, the tank will be designed in such way that the construction will allow reducing its capacity during non-vegetation period, e.g. by having multiple compartments that can be closed down if necessary.

Table 14. Proposed accumulation tank dimensions

ACCUMULATION TANK DIMENSIONS		TISOVEC	CHLUMĚTÍN	BOŘICE
volume small tank	[m^3]	3	2.5	2
total volume	[m^3]	12	10	8

Selected system for stormwater and treated wastewater reuse was **AS-RAINMASTER ECO 10**. According to the producer, this system is suitable also for greywater reuse. Effluent from MBRs has quality high enough to be used in this type of unit.

The energy consumption for potable water distribution and treatment in each municipality was estimated based on local situation. Tisovec is supplied from WTP Hamry [55]. The raw water is pumped from water reservoir on the river Chrudimka. Treatment technology consists of clarifiers followed by two-stage rapid sand filtration with activated carbon. The quality of water is quite low, due to low mineral content the water is aggressive and causes problems with corrosion and nitrite content.

In Chlumětín, water from the groundwater source is pumped into water storage tank and from there supplied by gravity. High-quality water does not need treatment and is only disinfected.

Municipality Bořice does not have a public water supply, water is supplied individually by wells.

For Tisovec and Chlumětín, the energy consumption was assumed to be equal to the national average based on data from Veolia [97]. The consumption in Bořice was calculated based on data by pump producers. Selected water pump model was SIGMA Darling Konta 80-2 (Q_{max} = 0.83 l/s, P = 1.5 kW) [81].

5.3 Operational reliability assessment

Reliability of a system refers to the ability to effectively fulfil its function for a specified period of time under specified conditions [61]. Reliability of a wastewater treatment system is expressed as percentage of time when the quality of the discharged effluent complies with the pre-defined emission limits. [2], [61] If the required emission limits are never exceeded, the system is 100% reliable. Reliability of a WWTP is directly related to effluent quality [61].

RELIABILITY OF SMALL-SCALE SYSTEMS

The reliability of small-scale plants was estimated based on expert elicitation of a Czech experienced practitioner, Ing. Karel Plotěný, who is a person authorised by Czech Ministry of Environment to carry out technical checks of on-site wastewater treatment systems (OZO). According to his experience as OZO, approximately one fourth of these systems do not work as expected when the first periodic inspection is carried out [68], i.e. within two years from installation of the system. However, when the second inspection is carried out, number of small-scale WWTPs discharging effluent non-compliant with legislative requirements is reduced to less than 5% [68].

If the local community in each municipality would agree on investments in series of small-scale WWTPs, these WWTPs could be remotely monitored by telemetry system which would in turn increase their reliability, as potential failure would get promptly detected. Expert elicitation estimates the failure rate of monitored WWTPs at 5% in the first year from installation, and at 1 to 2% in all following years [68].

According to Plotěný, the most reliable technology in general is a septic tank followed by soil filter, as it does not need extensive maintenance apart from occasional desludging and does not require energy inputs. Aerated bioreactors are second as far as reliability is concerned.

The most often encountered problems with ABs are related to longer inactivity periods, such as holidays etc. When there is a longer time when the WWTP is not in use, the organisms in the bioreactor begin to starve and die, and after wastewater starts flowing to the reactor again, it takes some time before the WWTP starts working properly again.

WWTPs that rely on electricity are capable of working for a certain time even in the case of power outage. Besides, power outages are nowadays rare and usually only short-term, therefore they are not considered as a serious factor affecting reliability, according to Plotěný.

RELIABILITY OF THE CENTRALISED OPTION

Reliability of a system can be analysed using various statistical methods, such as coefficient of variability (COR) [2], [61], probabilistic modelling, and generalised linear models. However, statistical methods require influent and effluent data.

For the purpose of this study, fault tree analysis (FTA) will be used to predict the reliability of WWTP Hrochův Týnec. For municipalities Tisovec and Chlumětín, system reliability will be calculated based on failure probabilities of individual elements of constructed wetlands and aerated ponds. The system reliability will be then predicted using the Monte Carlo simulation. The FTA will be not used for Chlumětín and Tisovec, because the centralised systems are natural-like treatment systems which makes the list of possible failure reasons very short. FTA analysis is more suitable for determining causal relationship between elements of more complicated systems, such as conventional mechanical-biological municipal WWTPs.

FTA is a deductive failure analysis based on breaking all the possible causes of failure into basic events whose probability can be estimated. These events are interrelated by logical causalities, and they result in one specific top event, for example exceeding a set limit. Probabilities of the basic events are estimated based on available data. This method was already used multiple times for

WTP reliability analysis (inter alia [47], [9]), and Taheriyoun et Moradinejad have used it also for reliability analysis of a WWTP in Tehran [86].

The advantage of this method is that it does not need extensive sets of data which are often difficult to obtain. The main limitation of FTA is that some factors that affect the overall reliability may not be detected. Another limitation is that failure probabilities of single basic events can be misjudged which then affects the overall reliability [86].

Reliability of a wastewater treatment system is defined as the ability of the system to produce effluent of given quality. Therefore, failure in this context means exceeding a given limit.

The Czech system of controlling effluent quality based on "p" and "m" values (described in Section 2.6) allows some samples to exceed the "p" value, while "m" value can never be exceeded. When average effluent concentrations c_{eff} are known, it can be assumed that the difference between c_{eff} and "p" is the maximum difference in both ways, e.g. that effluent during standard operating conditions would not have concentrations higher than "p" plus $(\mathrm{p} - c_{eff})$. Therefore, the threshold of failure for WWTP Hrochův Týnec was calculated as the difference between c_{eff} and "p" added to "p" value:

$$c_{fail} = p + \left(p - c_{eff}\right) = 2p - c_{eff} \tag{5.5}$$

The value c_{fail} for all investigated parameters fell within the range from "p" to "m", except for COD_{Cr}. For COD_{Cr} the calculated c_{fail} was higher than "m" = 130 mg/l, and therefore the failure threshold concentration for COD_{Cr} was considered to be equal to "m" value.

For Tisovec and Chlumětín, average effluent concentrations c_{eff} were not known, therefore a simplified assumption that the failure threshold will fall into range from "p" to "m" had to be made. Since the calculated c_{eff} for Bořice did not show any trends that could help estimate the relationship between c_{eff}, "p", and "m", the threshold concentration that is considered to be a sign of WWTP failure for Tisovec and Chlumětín was calculated as:

$$c_{fail} = \frac{p+m}{2}, \tag{5.6}$$

where "p" and "m" are respective emission standards for municipal WWTPs designed for less than 500 PE.

Concentrations of effluent when the WWTP fails would thus fall in the range from c_{fail} to $c_{influent}$. The assumed probability distribution of effluent concentration during failure is shown in Figure 17. Using random number simulations, first it was determined whether a failure will occur or not. In

case of failure, another random number was generated to simulate the seriousness of the malfunction. As seen in *Figure 17,* malfunctions were divided into five categories based on their severity, and the least severe breakdowns were given the highest probability. Therefore, what was defined as failure for the reliability analysis does not necessarily mean breaking the emission limits in reality, as concentration c_{fail} regarded as failure for the purpose of reliability assement was lower than value "m" for all cases, and values "p" can be exceeded to some extent according to Government Regulation No. 401/2015 Col. However, raised effluent concentrations above "p" limit indicate an unusual event that has caused the effluent quality to worsen.

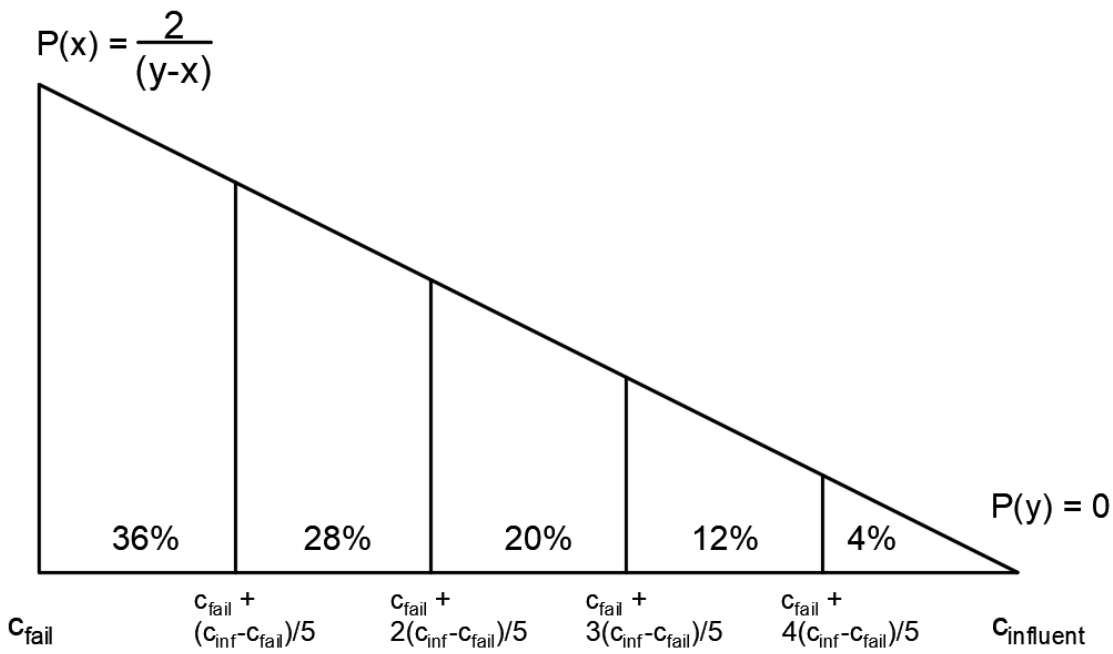

Figure 17. Distribution of concentration of samples that are above failure limit

6 RESULTS

6.1 Environmental assessment

6.1.1 Strictest scenario

This scenario compares daily pollution discharged to the environment based on the most stringent criteria for both centralised and decentralised option.

This scenario does not compare treatment efficiencies, because required efficiencies for both municipal WWTPs for agglomerations of less than 500 PE and for small-scale WWTPs falling under category III are the same, i.e. 85% reduction in BOD_5 and 75% reduction in COD_{Cr}. Instead, for municipal WWTP, "p" and "m" values were used to calculate the daily loads to receiving waters.

TISOVEC AND CHLUMĚTÍN

The results show that for municipalities Tisovec and Chlumětín, the centralised option would have smaller impact on the receiving waterbody in terms of BOD_5 and COD_{Cr} discharges. For Tisovec, the difference between daily loads to receiving waters is slightly more significant than for Chlumětín, i.e. the BOD_5 daily load is 2.5 times higher for the decentralised option in Tisovec, whereas in Chlumětín the BOD_5 load is 2.3 higher. This slight difference is caused by different assumptions of I/I in the network – in Chlumětín, the wastewater is assumed to be more dilute, which affects the daily loads to receiving waterbody calculated based on "p" and "m" values, while daily load calculated based on efficiency remain the same.

Table 15. Tisovec – strictest scenario

TISOVEC	production per day		BAT-AELs			WWTP cat. III	
	load	concentration	p	m	discharge	efficiency	discharge
PARAMETER	[kg/day]	[mg/l]	[mg/l]		[kg/day]	[%]	[kg/day]
BOD_5	21.78	579.7	30	50	1.31	85	3.27
COD_{Cr}	43.56	1159.4	110	170	4.70	75	10.89
TSS	19.97	531.4	40	60	1.69	-	-
TN	3.99	106.3	-	-	-	50	2.00
TP	0.91	24.2	-	-	-	80	0.18

It is worth mentioning that even though decentralised systems seem to be disadvantageous for Tisovec and Chlumětín in this scenario, their main asset may be imposition of removal requirements also for TN and TP, unlike BAT-AELs which only set emission standards for BOD_5, $COD_{Cr,}$ and TSS. However, water authority can set emission limits for municipal WWTP also for parameters for which

no emission standards are defined in Government Regulation No. 401/2015 Col. As stated before, all surface waterbodies in Czech Republic were designated as sensitive areas which are prone to eutrophication, more stringent restrictions on nutrient discharges are thus reasonable.

Table 16. Chlumětín – strictest scenario

CHLUMĚTÍN	production per day		BAT-AELs			WWTP cat. III	
	load	concentration	p	m	discharge	efficiency	discharge
PARAMETER	[kg/day]	[mg/l]	[mg/l]		[kg/day]	[%]	[kg/day]
BOD5	12.18	533.3	30	50	0.80	85	1.83
CODCr	24.36	1066.7	110	170	2.85	75	6.09
TSS	11.17	488.9	40	60	1.03	-	-
TN	2.23	97.8	-	-	-	50	1.12
TP	0.51	22.2	-	-	-	80	0.10

BOŘICE

As there is no suitable watercourse in Bořice that could be used for discharges of treated effluent, effluent from on-site WWTPs would have to comply with the requirements of Government Regulation No. 57/2016 which imposes more stringent regulations due to discharges to groundwater.

Table 17. Bořice - strictest scenario

BOŘICE	production per day		BAT-AELs			WWTP groundwater	
	load	concentration	p	m	discharge	efficiency	discharge
PARAMETER	[kg/day]	[mg/l]	[mg/l]		[kg/day]	[%]	[kg/day]
BOD_5	9.66	579.7	30	50	0.58	95	0.48
COD_{Cr}	19.32	1159.4	110	170	2.08	90	1.93
TSS	8.86	531.4	40	60	0.75	-	-
TN	1.77	106.3	-	-	-	50	0.89
TP	0.40	24.2	-	-	-	40	0.24

The results for Bořice show that in terms of pollution loading, the differences between centralised and decentralised option for daily pollution loads are negligible. Nevertheless, pollution discharged into groundwater is more controlled and restricted, as groundwaters are generally more vulnerable. Rivers and stream have a self-purification capacity that helps them to cope with smaller pollution loads, and pollution load that is no disturbance to a river can have long-term impacts on groundwater aquifer.

6.1.2 Tolerant scenario

The assumption of this scenario is that the least stringent regulations will be applied to discharges. For centralised scenario, emission limits will be set as the highest possible values, i.e. emission standards. Again, required efficiencies for BOD_5 and COD_{Cr} are identical for municipal WWTPs for agglomerations of less than 500 p.e and for small-scale WWTPs falling under category I. For BOD_5, the least strict requirement is 80% removal, for COD_{Cr} 70%. Therefore, for municipal WWTPs, values "p" and "m" will be used. For decentralised option, criteria for small-scale WWTPs falling under category I will be used for Tisovec and Chlumětín, and criteria for small-scale WWTPs discharging into groundwater will be used for Bořice.

The comparison showed that for discharges into surface waters, centralised WWTP would have smaller impact on the environment in terms of pollution discharges. For discharges into groundwater (Bořice), the decentralised option would result in smaller quantities of pollution being discharged into the environment.

TISOVEC AND CHLUMĚTÍN

The results for Tisovec and Chlumětín are similar to best-case scenario – centralised option would discharge less polluted effluent. In Tisovec, small-scale WWTPs would discharge 4.36 kg of pollution expressed as BOD_5 per day in total, whereas municipal WWTP would discharge only 43% of this amount.

Table 18. Tisovec - tolerant scenario

TISOVEC	production per day		emission standards			WWTP cat. I	
	load	concentration	p	m	discharge	efficiency	discharge
PARAMETER	[kg/day]	[mg/l]	[mg/l]		[kg/day]	[%]	[kg/day]
BOD_5	21.78	579.7	40	80	1.88	80	4.36
COD_{Cr}	43.56	1159.4	150	220	6.29	70	13.07
TSS	19.97	531.4	50	80	2.16	-	-
TN	3.99	106.3	-	-	-	-	-
TP	0.91	24.2	-	-	-	-	-

In Chlumětín, small-scale WWTPs discharge 2.1 times more BOD_5 per day than municipal WWTP. Discharges compliant with minimum efficiency requirements for small-scale WWTPs produce 2.44 kg BOD_5 per day, whereas application of emission standards results in daily load to receiving environment 1.14 kg BOD_5.

Table 19. Chlumětín - tolerant scenario

CHLUMĚTÍN	production per day		emission standards			WWTP cat. I	
	load	concentration	p	m	discharge	efficiency	discharge
PARAMETER	[kg/day]	[mg/l]	[mg/l]		[kg/day]	[%]	[kg/day]
BOD_5	12.18	533.3	40	80	1.14	80	2.44
COD_{Cr}	24.36	1066.7	150	220	3.83	70	7.31
TSS	11.17	488.9	50	80	1.31	-	-
TN	2.23	97.8	-	-	-	-	-
TP	0.51	22.2	-	-	-	-	-

BOŘICE

Centralised option seems to be less favourable for Bořice, where small-scale WWTPs would produce effluent of higher quality as compared to municipal WWTP, due to stringent efficiency requirements for discharges into groundwater. In terms of BOD_5, decentralised option reaches only 58% of pollution coming from municipal WWTP. However, restrictions of groundwater discharges are more stringent due to possible greater environmental impacts that can be caused by contaminating groundwater. It is therefore difficult to conclude which option would have smaller impact – whether discharging smaller pollution loads to more sensitive groundwater environment, or larger pollution loads to a river with a certain assimilative capacity.

Table 20. Bořice - tolerant scenario

BOŘICE	production per day		emission standards			WWTP groundwater	
	load	concentration	p	m	discharge	efficiency	discharge
PARAMETER	[kg/day]	[mg/l]	[mg/l]		[kg/day]	[%]	[kg/day]
BOD_5	9.66	579.7	40	80	0.83	95	0.48
COD_{Cr}	19.32	1159.4	150	220	2.79	90	1.93
TSS	8.86	531.4	50	80	0.96	-	-
TN	1.77	106.3	-	-	-	50	0.89
TP	0.40	24.2	-	-	-	40	0.24

6.1.3 Declared efficiency scenario

The goal of this scenario is to evaluate effluent quality that is achievable under normal operating conditions for both centralised and decentralised option.

TISOVEC

In Tisovec, the proposed centralised option is a constructed wetland (CW). Data on overall treatment performance have been obtained from monitoring data of an experimental three-stage constructed wetland system collected over two years of operation. The wetland was operated in

Czech conditions and consisted of saturated vertical-flow CW, free-drained vertical-flow CW, and horizontal-flow CW in series [99]. Average removal efficiencies for individual parameters are shown in *Table 21.*

When considering overall treatment efficiencies, decentralised options reach higher removal of BOD_5 and COD_{Cr} than multi-stage constructed wetland. The BOD_5 and COD_{Cr} daily load to receiving waters from CW was higher than daily load from each considered decentralised system, considering both tested and guaranteed values.

Almost all the decentralised systems showed better TSS removal performance than CW, apart from aerated bioreactor, based both on tested and guaranteed efficiencies.

As far as $N\text{-}NH_4$ removal is concerned, the daily load to receiving waters from decentralised systems ranges from 0.08 to 0.61 kg/day, whereas the daily load from CW is 0.28 kg/day. Sequencing batch reactor and systems with membrane filtration exhibit better $N\text{-}NH_4$ removal than CW, while aerated bioreactor (with or without advanced P removal) is less suitable for removing $N\text{-}NH_4$ than CW, based on tested values. $N\text{-}NH_4$ removal calculated based on guaranteed values is significantly lower than for CW, in some cases the daily load from a decentralised system is even twice as high.

Table 21. Tisovec - declared efficiency scenario (SBR - sequencing batch reactor; AB - aerated bioreactor; AB + APR - anaerobic bioreactor with advanced phosphorus removal; MBR - membrane bioreactor; AB ASIO - ASIO-VARIOcomp; AB + SF – ASIO-VARIOcomp + vertical soil filter AS-ZEON; ST + SF – septic tank AS-ANASEP + vertical soil filter AS-ZEON; MBR ASIO – ASIO-VARIOcomp ULTRA)

TISOVEC		PARAMETER	BOD_5	COD_{Cr}	TSS	TN	$N\text{-}NH_4$	TP
pollution generation	load	[kg/day]	21.8	43.6	20.0	4.0	2.5	0.9
	concentr.	[mg/l]	579.7	1159.	531.4	106.3	67.6	24.2
constructed wetland	removal eff.	[%]	92.5	83.8	96	79.9	88.8	30.0
	discharge	[kg/day]	1.63	7.06	0.80	0.80	0.28	0.64
Discharge calculated from tested efficiencies of small-scale WWTPs	SBR	[kg/day]	0.44	3.05	0.80	-	0.15	0.28
	AB	[kg/day]	0.44	3.92	1.40	-	0.30	0.55
	AB + APR	[kg/day]	0.22	2.18	0.40	-	0.58	0.06
	MBR	[kg/day]	0.00	1.74	0.00	-	0.08	0.39
discharge calculated from guaranteed ASIO values	AB ASIO	[kg/day]	1.08	3.94	1.22	-	0.61	0.32
	AB + SF	[kg/day]	0.23	2.82	0.14	-	0.61	0.32
	ST + SF	[kg/day]	0.23	1.78	0.47	-	0.47	0.24
	MBR ASIO	[kg/day]	1.08	3.94	0.47	-	0.47	0.32

Phosphorus removal in small-scale systems can be as high as 93% for systems with advanced phosphorus removal. Overall treatment efficiency for CW is only 30% which is less than for any of the tested WWTP. The daily loads to receiving waters from CW were 0.64 kg/day which is at least

twice as high compared to daily loads calculated based on guaranteed values, ranging from 0.24 to 0.32 kg/day. Tested WWTPs discharged 0.06 to 0.55 kg BOD_5/day, depending on the technology.

In conclusion, decentralised option would have smaller impact on receiving waterbody in Tisovec. All compared decentralised technological systems produced lower daily loads of BOD_5, COD_{Cr} and TP. Decentralised systems also reached better effluent quality than CW in terms of TSS, except for aerated bioreactor. CWs performed well in removing N-NH_4. CWs had an overall efficiency of N-NH_4 removal of 88.8%, and the daily load to receiving waters was lower than for most decentralised systems, except for SBR and MBR, when considering efficiencies achieved in testing at TGM WRI.

CHLUMĚTÍN

The proposed centralised option for Chlumětín is series of aerated stabilisation ponds. Overall treatment efficiencies presented in literature vary significantly, and the process efficiency can be influenced by many design factors – number of ponds, their size and shape, means of aeration, retention time, hydraulic and pollution load, and exogenous factors – climatic conditions, phytoplankton, daily and seasonal changes, maintenance quality, or system aging.

For example, in terms of BOD_5 removal, Effenberger and Duroň mention efficiencies ranging from 35 to 40% [82] as cited in [83] , while Just refers to values between 70 and 80% [40]. According to research conducted by Wanner, the average treatment efficiency is 65% [101].

Nutrient removal in stabilisation ponds can be 0 – 60% for TN, and 30 - 80% for TP. [101] For the comparison, overall efficiencies based on data obtained from monitoring six stabilisation ponds over 3 years were used. The monitored sample of stabilisation ponds included both aerated and non-aerated ponds, with or without mechanical pre-treatment, and with one pond or two in series. Given that for Chlumětín the proposed solution is a series of three aerated ponds, it can be assumed the efficiencies would be probably higher in real life.

The wastewater will mostly undergo primary treatment in septic tanks before it will be treated in the stabilisation pond. Since septic tanks usually remove about 50% of TSS and 30 – 40% of BOD_5 [91], the influent BOD_5 concentration to the WWTP would be around 348 – 406 mg/l BOD_5.

According to a study on low-loaded stabilisation ponds where the average influent BOD_5 concentration was 258 mg/l, overall system treatment efficiency for BOD_5 was 97,9%. [102] However, in this case, stabilisation ponds were used for tertiary treatment of effluent from a mechanical-biological WWTP, and the influent concentration for the study was only 63 – 75% of the calculated concentration in Chlumětín.

In reality, the efficiency would be likely higher than efficiencies achieved during the monitoring of six stabilisation ponds for untreated or mechanically pre-treated wastewater. However, the proposed solution would not probably achieve efficiencies as high as system of mechanical-biological WWTP followed by stabilisation ponds for tertiary treatment. Thus, this scenario compares results from both studies to achieved overall efficiencies of small-scale WWTPs and guaranteed values.

As the efficiencies of stabilisation ponds for untreated wastewater (in *Table 22* described *as aerated pond*) and for low-loaded stabilisation ponds for tertiary treatment (in *Table 22* described *as low-loaded pond*) vary quite significantly, and the precise efficiency for the proposed system in Chlumětín is not known, a definitive assessment cannot be made.

In general, low-loaded ponds achieve similar results to small-scale WWTPs when BOD_5 and COD_{Cr} are considered. It must be again mentioned that data for overall efficiencies of low-loaded stabilisation ponds show efficiencies of a system consisting of mechanical-biological WWTP + 2 stabilisation ponds, whereas the proposed solution in Chlumětín are septic tanks + 3 stabilisation ponds. BOD_5 removal of small-scale systems ranges from 0 to 0.24 kg BOD_5/day based on tested values, and from 0.14 to 0.66 kg BOD_5/day based on guaranteed values. Low-loaded ponds remove 97.9% of BOD_5 and produce a daily load to receiving waters of 0.26 kg/day. Aerated ponds would discharge 4.26 kg BOD_5 per day.

Table 22. Chlumětín - declared efficiency scenario (SBR - sequencing batch reactor; AB - aerated bioreactor; AB + APR - anaerobic bioreactor with advanced phosphorus removal; MBR - membrane bioreactor; AB ASIO - ASIO-VARIOcomp; AB + SF – ASIO-VARIOcomp + vertical soil filter AS-ZEON; ST + SF – septic tank AS-ANASEP + vertical soil filter AS-ZEON; MBR ASIO – ASIO-VARIOcomp ULTRA)

CHLUMĚTÍN		PARAMETER	BOD_5	COD_{Cr}	TSS	TN	$N\text{-}NH_4$	TP
pollution generation	load	[kg/day]	12.2	24.4	11.2	2.2	1.4	0.5
	concentr.	[mg/l]	533.3	1066.	488.9	97.8	62.2	22.2
aerated pond	efficiency	[%]	65	40	29	43.0	38	41.0
	discharge	[kg/day]	4.26	14.62	7.93	1.27	0.88	0.30
low-loaded pond	efficiency	[%]	97.9	93.7	93.8	77.4	99.6	95.6
	discharge	[kg/day]	0.26	1.53	0.69	0.50	0.01	0.02
tested small-scale WWTPs efficiencies	SBR	[kg/day]	0.24	1.71	0.45	-	0.09	0.16
	AB	[kg/day]	0.24	2.19	0.78	-	0.17	0.31
	AB + APR	[kg/day]	0.12	1.22	0.22	-	0.33	0.04
	MF	[kg/day]	0.00	0.97	0.00	-	0.04	0.22
guaranteed values ASIO	AB ASIO	[kg/day]	0.66	2.40	0.74	-	0.37	0.19
	AB + SF	[kg/day]	0.14	1.71	0.09	-	0.37	0.19
	ST + SF	[kg/day]	0.14	1.08	0.29	-	0.29	0.15
	MF ASIO	[kg/day]	0.66	2.40	0.29	-	0.29	0.19

The COD_{Cr} daily loads to receiving waters from decentralised systems ranged from 0.97 to 2.19 kg/day based on tested efficiencies. Daily load from low-loaded pond of 1.53 kg/day falls within this range. Aerated ponds produce 14.62 kg COD_{Cr} per day which is approximately 6 times more than from the least efficient decentralised system – aerated bioreactor.

Decentralised systems generally performed well in removing solids. All small-scale systems except AB produced lower daily loads than both low-loaded and aerated ponds, ranging from 0 to 0.45 kg TSS/day. Aerated bioreactors discharged daily loads of 0.78 and 0.74 based on tested and guaranteed values, respectively. Daily TSS load to receiving waters from low-loaded ponds was 0.69 kg/day, whereas aerated ponds that had overall removal efficiency 29% discharged 7.93 kg/day.

As far as nutrient removal is concerned, low-loaded ponds had 77.4% TN removal efficiency, and produced 0.50 kg of nitrogen daily. Aerated ponds discharged 1.27 kg TN/day and had an overall efficiency of 43%.

Low-loaded ponds removed nearly all ammonium (99.6%) and discharged 0.01 kg $N\text{-}NH_4$ per day. Daily load to receiving waters from small-scale systems ranged from 0.04 to 0.33 kg/day based on tested values, and 0.29 to 0.34 kg/day based on guaranteed values. Aerated ponds for secondary treatment released 0.88 kg $N\text{-}NH_4$ daily.

Low-loaded ponds produced effluent of highest quality by removing 95.6% of TP. Daily phosphorus load to receiving waters was 0.02 kg/day. Aerated bioreactors with advanced phosphorus removal reached comparable effluent quality. With an overall efficiency of 93%, series of ABs would discharge 0.04 kg/day. Other small-scale systems produced daily loads varying from 0.16 to 0.31 kg/day. Aerated ponds produced 0.30 kg of phosphorus daily.

In conclusion, small-scale systems and low-aerated ponds showed comparable performance for all the parameters. Aerated ponds discharged effluent of significantly lower quality in terms of BOD_5, COD_{Cr} and $N\text{-}NH_4$. Low-loaded ponds released lowest concentrations of TP. Decentralised systems discharged 0.04 – 0.31 kg phosphorus per day based on used technology. P removal efficiency of aerated ponds was comparable with the upper range of small-scale WWTPs.

Effluent quality from low-loaded ponds fell within the range of effluent quality from multiple types of small-scale systems for BOD_5, COD_{Cr} and TSS. Low-loaded ponds were even capable of removing more nutrients than small-scale systems. However, considered efficiencies for low-loaded ponds must be regarded as very optimistic because the ponds were used for less concentrated wastewater as tertiary treatment, whereas in Chlumětín the proposed system is septic tanks as primary treatment and stabilisation pond as secondary treatment. It is reasonable to conclude that

decentralised systems would produce less pollution in general, because the efficiency of stabilisation ponds in Chlumětín would be lower than for low-loaded ponds, and thus in most cases lower than for decentralised solutions.

BOŘICE

Centralised wastewater management in Bořice envisages construction of a separate sewage network that will transport wastewater to WWTP Hrochův Týnec. To compare the environmental impact of centralised and decentralised option, average efficiencies were calculated based on annual inflow load to WWTP and outflow load to receiving waterbody data from 2016, taken from TGM WRI Hydroecological Information System (HEIS) [98].

The calculated overall efficiencies are rather low, especially for nutrient removal. For size category of municipal WWTP 2,001 – 10,000 PE, required phosphorus removal efficiency is 70% according to Government Regulation No. 401/2015 Col, whereas the data show actual efficiency only 58.8%. However, a closer look at outflow concentrations show that they are in compliance with the emission limits set by the local authority, and therefore the low efficiency is caused mostly by low inflow concentrations.

For example, the average TP inflow concentration is 5.20 mg/l, whereas the calculated concentration of wastewater generated in Bořice is 28 mg/l. Such a low inflow concentration can be caused for example by large infiltration/inflow. This assumption can be supported also by data from PRVKÚK Pardubice which states overall parameters of wastewater prior to construction of existing WWTP Hrochův Týnec, when the wastewater was discharged straight into receiving water body (river Novohradka). According to PRVKÚK, average concentrations measured at junction of sewage network with Novohradka were as follows: BOD_5 43 mg/l, COD_{Cr} 124 mg/l, TSS 35 mg/l, TN 30 mg/l, $N\text{-}NH_4$ 22 mg/l, and TP 4 mg/l. Concentrations from 2016 are higher than concentrations listed in PRVKÚK, however, they are still low when considering annual inflow to WWTP and inflow pollution loads.

Table 23. WWTP Hrochův Týnec – inflow and outflow pollution load and concentration, efficiency, emission limits [98]

WWTP Hrochův Týnec	inflow to WWTP		discharge to water		efficiency	emission limits	
	[t/y]	[mg/l]	[t/y]	[mg/l]	[%]	p [mg/l]	m [mg/l]
BOD_5	12.76	118.40	0.63	5.84	95.1	25	50
COD_{Cr}	28.73	266.50	3.16	29.30	89.0	90	130
TSS	6.81	63.20	0.62	5.73	90.9	30	60
$N\text{-}NH_4$	4.66	43.20	1.01	9.40	78.2	15	30
TP	0.56	5.20	0.23	2.14	58.8	3	6

As the proposed separate sewage network would be connected to the existing network, it is likely that wastewater from Bořice would be also more dilute than previously estimated. Since there is no simple way of estimating the infiltration/inflow, and wastewater in the sewer undergoes various chemical and microbial transformations which affect its quality, for this scenario it was decided to compare environmental impact based on effluent concentrations, instead of daily loads to receiving waters. It was assumed that connection of wastewaters from Bořice would not affect the efficiency of WWTP Hrochův Týnec.

The results show that effluent discharged from WWTP Hrochův Týnec has higher BOD_5, COD_{Cr} and TSS concentrations than effluent discharged from decentralised systems. For BOD_5, effluent concentrations based on tested efficiencies of decentralised systems varied from 0 (MBR) to 13.3 mg/l (SBR and AB). Effluent concentrations of decentralised systems varied quite significantly depending on whether tested or guaranteed values were considered. For example, tested MBRs had 100% removal efficiency of BOD_5 and TSS, while the guaranteed value for MBR ASIO VARIOcomp ULTRA was 25 mg/l BOD_5 and 10 mg /l TSS.

Table 24. Bořice – declared efficiency scenario (WWTP HT – municipal WWTP Hrochův Týnec; SBR - sequencing batch reactor; AB - aerated bioreactor; AB + APR - anaerobic bioreactor with advanced phosphorus removal; MBR - membrane bioreactor; AB ASIO - ASIO-VARIOcomp; AB + SF – ASIO-VARIOcomp + vertical soil filter AS-ZEON; ST + SF – septic tank AS-ANASEP + vertical soil filter AS-ZEON; MBR ASIO – ASIO-VARIOcomp ULTRA)

BOŘICE		PARAMETER	BOD_5	COD_{Cr}	TSS	$N\text{-}NH_4$	TP
pollution generation	load	[kg/day]	9.7	19.3	8.9	1.1	0.4
	concentration	[mg/l]	667	1333	611	78	28
WWTP HT	concentr.	[mg/l]	28.62	127.4	48.20	14.71	9.95
tested small-scale WWTPs efficiencies	SBR	[mg/l]	13.33	93.33	24.44	4.67	8.61
	AB	[mg/l]	13.33	120.0	42.78	9.33	16.94
	AB + APR	[mg/l]	6.67	66.67	12.22	17.89	1.94
	MBR	[mg/l]	0.00	53.33	0.00	2.33	11.94
guaranteed values ASIO	AB ASIO	[mg/l]	25	90	30	15	8
	AB + SF	[mg/l]	5	70	3	15	8
	ST + SF	[mg/l]	5	40	10	10	6
	MBR ASIO	[mg/l]	25	90	10	10	8

$N\text{-}NH_4$ removal in WWTP Hrochův Týnec was comparable to AB and AB combined with a soil filter, according to guaranteed values. Effluent from small-scale systems calculated based on tested efficiencies had better quality than effluent from WWTP Hrochův Týnec, except for AB with advanced phosphorus removal.

All decentralised systems except for AB and MBR (tested values) provided slightly higher phosphorus removal as compared to WWTP Hrochův Týnec.

6.1.4 Environmental assesment summary

The purpose of this assessment was to compare centralised and decentralised management of wastewater treatment in terms of pollution that is discharged to the environment.

The assessment confirmed that Czech legislative imposes more stringent requirements for municipal WWTPs (50-500 PE) than for small-scale systems discharging into surface waters (< 50 PE). The strictest scenario compared the most stringent requirements that can be imposed by water authority, and the resulting concentrations discharged from small-scale systems were at least twice as high in terms of BOD_5 and COD_{Cr} for both Tisovec and Chlumětín.

Requirements for groundwater discharges are more stringent than those for surface waters. Decentralised wastewater management in Bořice would produce comparable pollution loads to receiving environment as centralised WWTP, according to the strictest scenario.

Tolerant scenario investigated environmental impact of wastewater treatment when the highest permissible concentrations are allowed. For discharges into surface waters, centralised management would produce less pollution than decentralised systems, similarly to the strictest scenario. If requirements for groundwater discharges were applied and compared to centralised WWTP, the daily loads to receiving environment from decentralised systems would be lower.

However, when considering concentrations that are achievable under normal operating conditions, decentralised options produced effluent of higher quality than centralised option. Declared efficiency scenario showed that effluent from decentralised systems contains less pollution than effluent from centralised WWTPs, although the legislative requirements are apparently less demanding for on-site systems.

6.2 Energy assessment

6.2.1 Energy requirements for wastewater treatment – centralised WWTP compared to small-scale systems

TISOVEC

Proposed centralised option for Tisovec is a constructed wetland located in Dřeveš. Energy requirements for constructed wetlands vary depending on wetland type, area of the wetland, treated volume per day, etc. Luederitz et al. calculated average energy consumption for CW operation on semi-centralised scale as 135 MJ/PE/annum, which equals to 0.1 kW/PE/day.

In addition, some energy will be required for transport of the sewage to Dřeveš. Wastewater from local parts Dřeveš and Otáňka can be transported by gravity as the slope of the terrain is sufficient. Wastewater from Kvasín is transported by gravity to a pumping station which then pumps it above Vrbětice. Wastewater from both Kvasín and Vrbětice is then transported by gravity to a pumping station, and then pumped to the CW. Wastewater from local part Tisovec flows by gravity to a pumping station and then is pumped to the CW. Assumed I/I was 15%. Schematic positioning of sewage network reaches is shown in *Figure* 18.

Figure 18. Schematical situation of proposed sewage network in municipality Tisovec (continuous line - gravity sewer, dashed line - pressure sewer (map taken from map server www.mapy.cz)

In total there will be four submersible sludge pumps. For pumping wastewater from Vrbětice and Tisovec, pump type SP 750 F Extol Premium 8895001 (Q_{max} = 0.60 l/s, H_{max} = 10 m; P = 0.75 kW) [82]will be used. To transport wastewater from Kvasín to Vrbětice, two pumping stations will be necessary due to elevation difference between Kvasín and Vrbětice which is almost 40 metres from

the lowest point in Kvasín. Selected pumping stations are type SIGMA 40-GFZU (MH) 400V (Q_{max} = 4.8 l/s, H_{max} = 19 m; P = 1.5 kW) [81]. *Table 23* summarises energy requirements for pumping.

Table 25. Energy requirements for pumping - Tisovec

municipalities	PE	volume pumped [m^3/day]	Q_{max} [l/s]	pumping [h/day]	[kW]	energy [kWh/day]
Kvasín	73	7.6	4.8	0.44	1.5	0.33
Kvasín + Vrbětín	125	20.5	0.6	9.49	0.75	7.12
Tisovec	84	8.7	0.6	4.03	0.75	3.02
					TOTAL	**10.79**

The comparison showed that CWs have significantly lower energy requirements than any decentralised system. A centralised CW would consume approximately 27 kWh/day, while completely decentralised system consisting of 52 ABs would consume 50 kWh/day. If the selected technology was SBR, the total consumption would be 251 kWh/day if every household had a SBR. Series of MBRs would require daily input of 229 kWh.

Not surprisingly, aerated bioreactors consume the least amount of energy of all three compared decentralised systems, while generally producing effluent of poorest quality as shown in environmental assessment section. The results also show that the energy consumption would be lower in case of semi-centralised solution. For example, if every household in Tisovec had its own MBR, there would be 73 WWTPs with total energy consumption of 229 kWh per day.

However, implementation of multiple larger MBRs to form cluster systems for up to 24 EO would reduce the total daily consumption to 127 kWh/d. As MBRs generally provide effluent of highest quality (in terms of BOD_5, COD_{Cr}, TSS, and N-NH_4), implementation of cluster systems would be beneficial because it would have lower energy requirements than decentralised option with SBR while releasing lower daily loads of pollution to receiving waterbody.

Table 26. Tisovec- energy assessment

TISOVEC								
constructed wetland			SBR		MBR		AB	
	[kWh/PE/yr]	[kWh/d]	[no. of systems]	[kWh/d]	[no. of systems]	[kWh/d]	[no. of systems]	[kWh/d]
operation	37.5	16.54	121	250.5	73	228.7	52	49.8
pumping	24.5	10.79	73	150.3	36	129.6	36	43.6
total	-	**27.33**	52	107.3	21	174.9	28	63.7
-	-	-	36	91.8	15	127.1	21	38.4
-	-	-	-	-	-	-	15	34.5

CHLUMĚTÍN

Aerated stabilisation pond, proposed as centralised solution in Chlumětín, has energy consumption in the range from 0.16 to 0.97 kWh/m^3, with median value 0.46 kWh/m^3 [26], according to data gathered from 369 stabilisation ponds with various technological setting located in Europe and North America.

Table 27 shows that as far as energy consumption is concerned, aerated ponds are more suitable than on-site systems. Energy consumption of aerated ponds ranges from 3 to 18 kWh/day, while system of ABs would consume 19 to 28 kWh/day, depending on the centralisation rate. Completely decentralised system of 68 SBRs would require energy input of 140 kWh per day, which is about 17 times more than centralised option (median value.) Semi-centralised system consisting of 8 MBRs would consume about 8 times more energy than the stabilisation pond (median value).

Table 27. Chlumětín - energy assesment

CHLUMĚTÍN								
aerated pond			SBR		MBR		AB	
[kWh/m^3]		[kWh/d]	[no. of systems]	[kWh/d]	[no. of systems]	[kWh/d]	[no. of systems]	[kWh/d]
min	0.16	2.92	68	140.1	41	127.9	29	27.8
median	0.46	**8.40**	41	84.0	20	72.5	20	24.4
max	0.97	17.72	29	60.0	12	97.8	16	35.6
-		-	20	51.4	8	71.1	12	21.5
-		-	-	-	-	-	8	19.3

BOŘICE

Centralised wastewater option is to connect Bořice to an existing plant in Hrochův Týnec. Energy consumption of a municipal WWTP depends on many factors, inter alia volumes treated, used technology, required quality of effluent, age of mechanical equipment and implementation of possible energy recovery strategies, such as electricity generation in CHP units. Generally, it can be said that larger WWTPs have lower unit energy consumption, as reflected also in analysis of energy consumption of 1777 WWTPs from 2005 [7]. WWTPs are divided into five size categories and the unit energy consumption is inversely proportional to the size, as shown *in Table 28.*

WWTP Hrochův Týnec falls under the category 2,001 – 10,000 EO, the energy consumption was thus estimated as 0.5 kWh/m^3. According to the application for a building permit submitted by municipality Bořice, wastewater would be transported by gravity and where necessary it would be pumped by two pumping stations over a total distance of approx. 1.5 km. Energy needed to operate these pumps was included in the calculation.

Table 28. Energy consumption per annum and unit energy consumption for Czech WWTPs [7]

size category	energy consumption	
	[MWh/yr]	[kWh/m^3]
< 500 EO	14294.6	0.7
501 - 2,000 EO	35778.6	0.6
2,001 - 10,000 EO	67715.8	0.5
10,001 - 100,000 EO	135125.1	0.4
> 100, 000 EO	68709.4	0.3

According to map data, the terrain in the area is flat and mildly sloped towards the existing WWTP. The average wastewater production per capita was taken as 90 l/PE/day and expected infiltration/inflow of the separate network was estimated at 15%, so the total inflow to the existing network would be approximately 0.17 l/s. Selected submersible sludge pump was SP 750 F Extol Premium 8895001 (Q_{max} = 0.60 l/s, H_{max} = 10 m; P = 0.75 kW). To pump wastewater from Bořice plus 15% I/I, the pump would have to operate almost 8 hours per day which would require 5.8 kWh/day.

Table 29. Bořice - energy assessment

BOŘICE							
municipal WWTP		SBR		MBR		AB	
	[kWh/d]	[no. of systems]	energy [kWh/d]	[no. of systems]	energy [kWh/d]	[no. of systems]	energy [kWh/d]
operation	7.25	54	111.1	32	101.4	23	22.1
pumping	5.79	32	66.7	16	57.5	16	19.3
total	**13.03**	23	47.6	9	77.6	12	28.2
	-	16	40.7	7	56.4	9	17.0
	-	-	-	-	-	7	15.3

The results show that energy consumption of a centralised plant is still the lowest between compared options even when considering energy consumption for pumping.

Daily consumption of a system of ABs ranges from 15 kWh/day for semi-centralised arrangement to 22 kWh/day for decentralised arrangement. Compared to municipal WWTP, SBRs consume 3 to 9 times more energy per day, depending on the centralisation rate – semi-centralised solution would have lower energy requirements as compared to fully decentralised implementation of SBRs for each household. MBRs consume over 4 times more energy than the municipal plant when implemented at semi-centralised scale. Decentralised MBRs consume almost 8 times more energy than the municipal WWTP per day.

6.2.2 On-site reuse systems – energy requirements for operation compared to energy that would be needed for treatment and distribution of the same volume of potable water

STORMWATER ACCUMULATION ONLY

The accumulation tank was designed for 28-day storage, thus average volume of water that would be pumped daily during the vegetation season was determined as $\frac{tank\ volume}{28\ days}$. RAINMASTER ECO can pump 10 l/min, so the unit would be consuming energy approximately 0.23 hour per day. With required energy input 90 W, one unit would consume only 0.021 Wh per day and household.

The results show that modern stormwater reuse systems have very low energy requirements and that daily energy consumption is minimal. For pumping approximately 140 litres daily, the selected system would consume 21 Wh per household, while average energy consumption of centralised WTP including distribution for the same volume of potable water are 74 Wh/d, and energy requirements for pumping potable water from wells are 71 Wh/d.

Table 30. Comparison of energy consumption of stormwater reuse and potable water treatment and distribution

municipality	STORMWATER			POTABLE WATER		
	stormwater available	stormwater reuse energy requirements	total energy	energy for potable water production		total energy
	[l/day/ household]	[Wh/day/ household]	[kWh/day]	[kWh/m^3]	[Wh/d/ household]	[kWh/day]
TISOVEC	142.86	21.4	2.21	0.523	74.7	7.70
CHLUMĚTÍN	142.86	21.4	1.86	0.523	74.7	6.50
BOŘICE	142.86	21.4	1.89	-	71.7	6.31

However, it needs to be highlighted that AS-RAINMASTER Eco 10 is targeted at energy saving and as such has significantly lower energy requirements and Q_{max} than other products on the Czech market. Examples of other stormwater-reusing systems offered on the Czech market and their energy consumption are listed in following table:

Table 31. Pumping systems for reuse of stormwater available on Czech market [107], [3], [73]

manufacturer	model	Q_{max}	P
		[l/min]	[W]
EASYPUMP	ECORAIN ADVANCED	85	1000
ASIO	AS-RAINMASTER FAVORIT 20	80	800
zehnder-pumpen	RWNA Eco Compact 11	48.3	600

Recalculating the energy consumption based on energy input required for AS-RAINMASTER FAVORIT 20 showed that energy spent on pumping stormwater would be approximately 2.6 times

higher than energy required for distribution and treatment of potable water. Nevertheless, AS-RAINMASTER Eco 10 is a functional technology that proves that stormwater reuse can be both environmentally conscious and economic.

STORMWATER AND TREATED WASTEWATER ACCUMULATION

As shown in the previous part of energy assessment, MBRs are generally the most energy-intensive from both decentralised and centralised technologies, followed by SBRs. However, MBRs offer the possibility to reuse the treated effluent for irrigation, toilet flushing, laundry washing, and other purposes that do not require potable water.

If treated wastewater of sufficient quality was available, it could be collected together with stormwater. Therefore, consumption of potable water could be further reduced.

The aim of this assessment is to investigate whether it would be efficient to collect and reuse stormwater or effluent from MBRs from energetic point of view, and if the energy spent on pumping stormwater and treated wastewater would be lower than energy required to treat and distribute potable water from a source.

Table 32. Comparison of energy consumption of stormwater and treated wastewater reuse and potable water treatment and distribution

municipality	STORMWATER			POTABLE WATER		
	non-potable water available	reuse energy requirements	total energy	energy for potable water production		total energy
	[l/day/ household]	[Wh/day/ household]	[kWh/day]	[kWh/m^3]	[Wh/d/ household]	[kWh/day]
TISOVEC	381.9	57.29	5.90	0.523	199.74	20.57
CHLUMĚTÍN	292.2	43.83	3.81	0.523	152.82	13.30
BOŘICE	246.9	37.03	3.26	-	123.92	10.91

The results showed that reusing both accumulated stormwater and effluent from MBRs is beneficial from energetic point of view, and can save up to 14.7 kWh/day during the vegetation period per household in Tisovec, where there are the most people living in one household (3.3 PE). Reusing stormwater and treated wastewater can cover 30 to 40 percent of potable water that is used for irrigation, flushing and laundry washing, and saves energy needed for treatment and distribution of potable water. In the period from November to March, requirements of water for non-potable purposes will be covered by treated effluent production. Smaller volume of water will be reclaimed, energy consumption of reuse systems is thus 1.6, 1.06 and 0.92 kWh/day, and corresponding energy consumption for centralised potable water treatment and distribution is 5.6, 3.7, and 3.1 kWh/day.

6.2.3. Energy consumed for decentralised wastewater and stormwater management versus centralised WTP and WWTP

MBRs consume most energy of all the decentralised technologies, but on the other hand offer the possibility of effluent reclamation for irrigation and household purposes, which can save energy for treatment of potable water. The aim of this assessment is to compare completely centralised distribution of potable water and subsequent disposal of wastewater in a centralised system to a completely decentralised MBR system with reuse of stormwater and treated effluent in term of energy requirements.

For the purpose of this assessment, it was assumed that the MBRs will be completely decentralised, i.e. the only the technological option of MBR for maximum capacity 5 PE was compared. Although MBRs on semi-centralised scale have lower energy requirements, treated wastewater reclamation is more feasible on completely decentralised scale. MBR AS-VARIOcomp K ULTRA can treat wastewater from 3 to 5 PE, consumes energy 21 hours per day [68] and requires energy input 150 W [4]. This assessment compares energy requirements for wastewater treatment, energy requirements for stormwater and treated effluent reuse with energy consumption for treatment and distribution of corresponding volume of potable water. However, it does not calculate requirements for potable water used for purposes where potable water cannot be replaced – the energy requirements will be the same for both centralised and decentralised approach. The comparison shows average energy consumption for vegetation period April – October.

Table 33. Energy consumption for decentralised system with MBRs and stormwater accumulation

DECENTRALISED	WASTEWATER		STORMWATER & TREATED WASTEWATER			TOTAL
municipality	no of MBRs	energy [kWh/d]	available V [l/day/household]	[Wh/day/ household]	energy [kWh/d]	energy [kWh/d]
TISOVEC	73	229	381.9	57.3	5.9	**235**
CHLUMĚTÍN	41	128	292.2	43.8	3.8	**132**
BOŘICE	32	101	246.9	37.0	3.3	**105**

Table 34. Energy consumption for centralised water and wastewater treatment system

CENTRALISED	WASTEWATER		POTABLE WATER			TOTAL
municipality	technology	[kWh/d]	[kWh/m^3]	[Wh/day/ household]	energy [kWh/day]	energy [kWh/d]
TISOVEC	CW	27.33	0.523	199.7	20.6	**47.9**
CHLUMĚTÍN	aerated pond	8.40	0.523	152.8	13.3	**21.7**
BOŘICE	mech-biol. WWTP	13.03	-	123.9	10.9	**23.9**

The results show that although treated wastewater and stormwater reuse systems save energy for potable water consumption, the energy demands of a MBRs are still significantly higher than for a centralised WWTP, and so decentralised systems with effluent and stormwater reuse consume approximately 4.4 to 6 times more energy than centralised option. Nevertheless, additional environmental benefits arising from saving potable water must be also considered.

6.2.4 Energy assessment summary

Centralised wastewater management proved to be less energy intensive as compared to decentralised systems. Centralised WWTPs benefit from economies of scale and so the unit energy consumption is usually lower than for decentralised systems. However, construction of sewage network often requires significant financial (and energetic) inputs that can make centralised systems prohibitively expensive to implement.

It was also shown that semi-centralised arrangement of on-site WWTPs would consume less energy than if every household would have its own domestic WWTP.

Decentralised systems can also have zero energy requirements – an example of such system is a septic tank for primary treatment followed by a soil filter. System ST+SF is capable of producing effluent quality comparable to other decentralised systems.

Comparison of systems for stormwater and treated wastewater reuse showed that modern devices for distributing water for reuse have very low energy requirements, making them favourable not only because they provide supply of water for non-sanitary and non-drinking purposes, but also from energetic point of view. Reuse systems could also help partly offset the financial burden of MBRs by reducing expenditure on potable water. Nevertheless, it was shown that decentralised system of MBRs has 4.5 to 6 times higher energy consumption than centralised WTP and WWTP, even if energy savings resulting from water reuse are considered.

6.3 Operational reliability assessment

BOŘICE

For WWTP Hrochův Týnec, the hierarchy of factors that cause failures leading to exceeding of emission limits is shown in *Figure 19*. There are seven basic events contributing to failures: operator absence (O_1), operator error (O_2), equipment failure (O_3), design problem (O_4), toxic entry (O_5), high infiltration/inflow (O_6), hydrometeorological factors (O_7). The symbol for “AND” gate is  , for

"OR" gate  . Gates show causal actions leading to failures. If two basic events are connected by an "AND" gate, both of these event need to happen at the same time for failure to occur. "OR" gate expresses that only one of the events is enough to trigger the failure. Possible failures of the mechanical-biological plant are divided into two main groups, failures in aeration tank, and failures in secondary sedimentation.

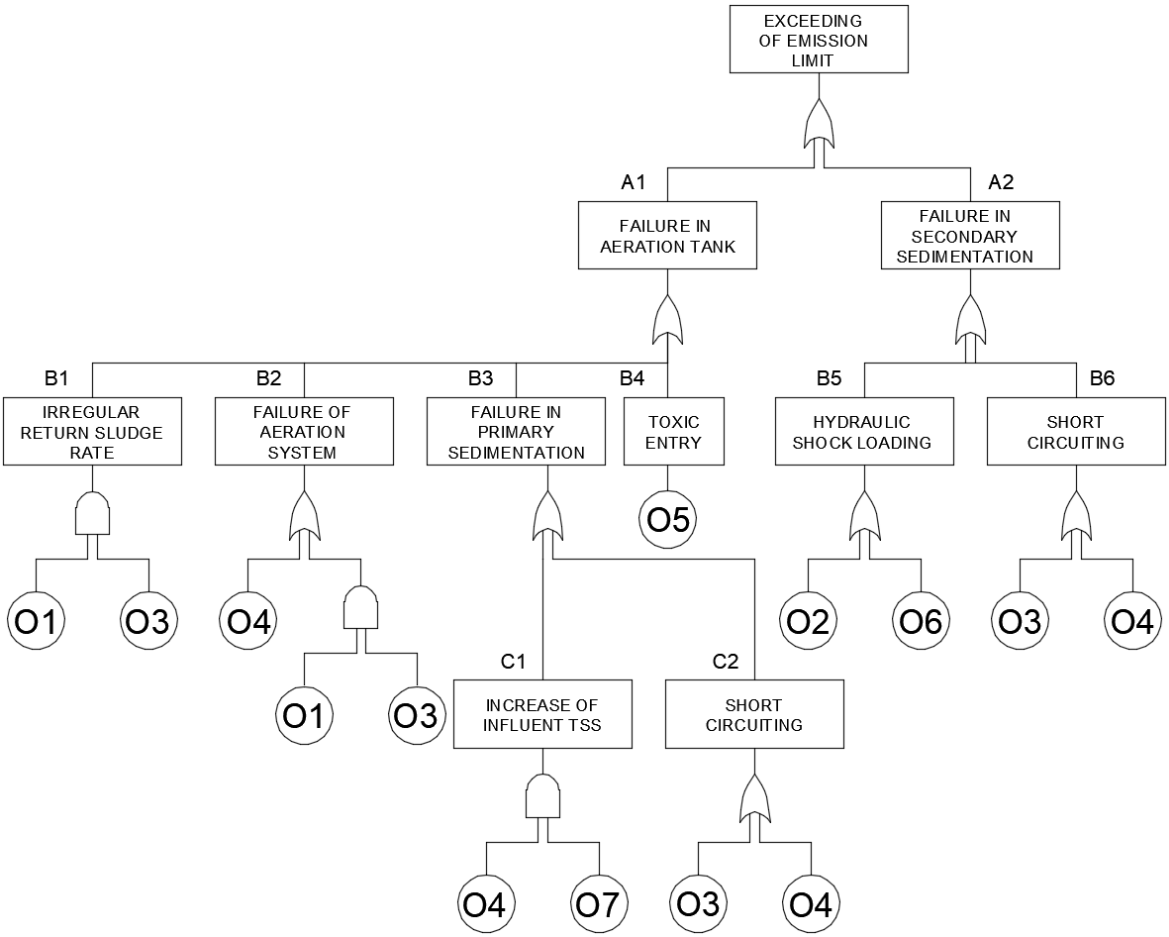

Figure 19. Fault tree - WWTP Hrochův Týnec (O1 - operator absence; O2 - operator error; O3 - equipment failure; O4 - design problem; O5 - toxic entry; O6 - high infiltration/inflow; O7 - hydrometeorological factors) (after [86]*)*

Failures in aeration tank can be caused by problems in primary sedimentation, malfunctions of aeration system, entries of substances toxic for the microorganisms, or by failures of return sludge system. Failures in secondary sedimentation is caused by short circuiting or by hydraulic shock loading, for example due to high I/I.

Following equations show the minimal cut set calculated using Boolean algebra ("∪" denotes "OR", "∩" denotes "AND"):

$$N = A_1 \cup A_2 \tag{6.1}$$

$$A_1 = B_1 \cup B_2 \cup \mathrm{B}_3 \cup B_4 \tag{6.2}$$

$$A_1 = (O_1 \cap O_3) \cup \big(O_4 \cup (O_1 \cap O_3)\big) \cup (O_4 \cap O_7) \cup O_3 \cup O_4 \cup O_5 = O_3 \cup O_4 \cup O_5 \tag{6.3}$$

$$A_2 = B_5 . B_6 = (O_2 \cup O_6) \cup (O_3 \cup O_4) = O_2 \cup O_3 \cup O_4 \cup O_6 \tag{6.4}$$

$$N = A_1 . A_2 = (O_3 \cup O_4 \cup O_5) \cup (O_2 \cup O_3 \cup O_6) = O_2 \cup O_3 \cup O_4 \cup O_5 \cup O_6 \tag{6.5}$$

Probabilities of basic events have been estimated based on consultations and literature search.

The chance that operator of the plant will cause a failure (operator error - O_2) was estimated as 0.5%. Probability of equipment failure (O_3) was estimated as 1.5%. Failures caused by definciencies in design (O_4) were expected to be 0.5%. Failures caused by presence of toxic substances in the influent to WWTP (O_5) were estimated as 2% [86]. Failures due to high infiltration/inflow (O_6) were expected to be 4%, as the sewage network in Hrochův Týnec shows signs of high I/I.

The probability of the top event was then determined as sum of basic events probabilities:

$$P(N) = P\,(O_2 \cup O_3 \cup O_4 \cup O_5 \cup O_6) = 0.005 + 0.015 + 0.005 + 0.02 + 0.04 = 0.085 \tag{6.6}$$

Thus, the reliability of the plant in Hrochův Týnec is:

$$reliability = 1 - P\,(N) = 1 - 0.085 = 0.915 \tag{6.7}$$

Using 10000 randomly generated combinations of basic events in Excel, quantitative analysis has been performed, using the Monte Carlo simulation. For each basic event, it was determined whether it will occur or not based on the randomly generated probability. The top event occurs when at least one basic event occurs. From all the simulations (N), the top event occurred 811 times (m). Thus, the probability was calculated as:

$$P_T = \frac{m}{N} = \frac{811}{10000} = 0.0811 \tag{6.8}$$

The reliability calculated using Monte Carlo simulation is 91.89%. To have a look at the importance of factors influencing reliability, probability of the top event was calculated for five other scenarios. In each scenario, one of the basic events was absent (e.g. it was assumed that one specific event could not occur.) As the minimal cut set of basic events is very simple, such that one basic event from the minimal cut set will always lead to failure, the calculated improvement factors (IFs) directly mirror the probability of a basic event. Therefore, the greatest impact on reliability have operator error (O_2) and high I/I (O_6).

Improvement factors were calculated as follows:

$$IF = \frac{P(N)-P(N_0)}{P(N)} \quad (6.9)$$

where P(N) is the probability of WWTP failure (top event) and $P(N_0)$ is the probability of failure when one of the basic events is omitted [86].

Table 35. Basic events - probability, P(N0) and improvement factor

symbol	basic event	P_n [%]	failures when O_n is omitted	$P(N_0)$	IF [%]
O_2	operator error	0.5	758	0.076	6.54
O_3	equipment failure	1.5	660	0.066	18.62
O_4	design problem	0.5	770	0.077	5.06
O_5	toxic entry	2	631	0.063	22.19
O_6	high I/I	4	442	0.044	45.50

Reliability assessment investigated the additional pollution that would be discharged to the receiving waters when failures of centralised and centralised systems would be taken into consideration.

Pollution discharged into the environment from centralised and decentralised systems during failure is summarised in *Table 36.* This assessment compares pollution loads discharged during non-standard situations, e.g. when the effluent concentrations are above failure threshold concentration c_{fail}. Pollution discharged under normal operating conditions is summarised in Chapter 6.1.3.

Table 36. Bořice - pollution discharged to the environment during WWTP failure

BOŘICE		PARAMETER	BOD_5	COD_{Cr}	TSS	$N\text{-}NH_4$	TP
pollution generation	load	[kg/day]	9.7	19.3	8.9	1.1	0.4
	concentratio	[mg/l]	579.7	1159.4	531.4	67.6	24.2
emission standards	p	[mg/l]	25	90	30	15	3
	m	[mg/l]	50	130	60	30	6
average c	c_{eff}	[mg/l]	5.84	29.30	5.73	9.40	2.14
c -failure threshold	$c_{fail} = 2p - c_{eff}$	[mg/l]	44.16	130.00	54.27	20.60	3.86
average c during failure		[mg/l]	227.91	223.96	475.62	214.46	36.39
WWTP Hrochův Týnec	volume	[m^3/day]	1.4	1.4	1.4	1.4	1.4
	load	[kg/day]	0.305	0.648	0.292	0.050	0.015
small-scale WWTPs	5% systems	[kg/day]	0.483	0.966	0.443	0.056	0.020
	1.5%	[kg/day]	0.145	0.290	0.133	0.017	0.006

The results of reliability analysis show that concerns about reliability of small-scale systems are valid for unmonitored decentralised systems. Approximately 5% of these systems does not provide sufficient treatment, and daily loads to receiving environment are significantly higher for BOD_5, COD_{Cr}, and TSS.

However, it was also shown that centralised monitoring can significantly improve the reliability of these systems, and that decentralised systems that are centrally monitored produce lower daily load of pollution to receiving environment as compared to centralised systems.

Estimated number of decentralised systems that do not function properly and discharge untreated effluent into the environment was 1.5%. In Bořice, if 1.5% of produced pollution were discharged untreated, it would represent 0.145 kg BOD_5 per day, whereas from Hrochův Týnec the daily pollution load from just from Bořice would be 0.298. As this WWTP treats wastewater from 2300 PE in total, the total discharged pollution would be way higher – the comparison calculates only with the pollution part produced in Bořice.

TISOVEC

Possible reasons for failure of a constructed wetland together with estimated probabilities are listed in *Table 37.* One of the most encountered problems with CWs in Czech Republic is their clogging and subsequent forming of preferential surface flow paths that reduces the efficiency because wastewater does now flow through the filter media [42], [58]. This can be prevented by sufficient mechanical primary treatment. The probability that portion of solids will enter the wetland and over time contribute to clogging was estimated at 4%. Wastewater is pumped into the wetland with a pump which can fail with estimated probability 1.5%. CWs are located outside and the processes can be affected by atmospheric conditions. Assumed probability of failure due to atmospheric conditions was 0.5%. Failures due to other design problems, such as hydraulic short-circuiting, were given the probability 0.5%.

Probability that the CWs will be unable to produce effluent of sufficient quality was calculated as sum of probabilities of each cause of failure:

$$P(N) = P_1 + P_2 + P_3 + P_4 = 0.04 + 0.015 + 0.005 + 0.005 = 0.065 \qquad (6.10)$$

The reliability of the CW is thus 93.5%. In the Monte Carlo simulation, failure occurred in 615 cases from total 10 000 cases. The probability of failure was thus predicted as $\frac{615}{10000} = 0.0615$, reliability as 93,85%.

Probabilities of CW failure when one cause of failure is omitted $P(N_O)$ were also calculated, and used to calculate the improvement factors. $P(N_O)$ and improvement factors (IFs) are shown in *Table 37*.

Table 37. Tisovec - possible causes of failure of a constructed wetland and respective probabilities, $P(N_0)$ and IFs

cause of failure		probability [%]	failures when omitting P_n	$P(N_0)$	IF [%]
P_1	clogging	4	246	0.0246	60.00
P_2	pump failure	1.5	466	0.0466	24.23
P_3	hydrometeorological conditions	0.5	579	0.0579	5.85
P_4	design problem	0.5	568	0.0568	7.64

Table 38 shows daily pollution loads to the receiving environment when failures of centralised constructed wetland and decentralised systems are taken into consideration. When considering 93.85% system reliability in Bořice, decentralised systems without monitoring result in almost 400 kilograms of BOD_5 per annum being discharged during non-standard situation into the receiving environment as compared to 183 kilograms BOD_5 from centralised systems. Implementation of centralised monitoring system reduces the annual additional pollution loads to 120 kg BOD_5 which makes centrally monitored decentralised systems the most reliable option in terms of BOD_5, COD_{Cr}, and TSS removal.

Table 38. Tisovec - pollution discharged to the environment during WWTP failure

TISOVEC		PARAMETER	BOD_5	COD_{Cr}	TSS	TN	N-NH_4	TP
pollution generation	load	[kg/day]	21.8	43.6	20.0	4.0	2.5	0.9
	concentr.	[mg/l]	579.7	1159.	531.4	106.3	67.6	24.2
emission standards	p	[mg/l]	150	40	50	-	-	-
	m	[mg/l]	220	80	80	-	-	-
c – failure threshold	c_{fail} = (p+m)/2	[mg/l]	185	60	65	-	-	-
average c during failure		[mg/l]	232.5	508.4	219.8	-	-	-
constructed wetland	volume	[m^3/day]	2.3	2.3	2.3	2.3	2.3	2.3
	load	[kg/day]	0.5	1.2	0.5	-	-	-
small-scale WWTPs	5% systems	[kg/day]	1.089	2.178	0.998	0.200	0.127	0.045
	1.5% systems	[kg/day]	0.327	0.653	0.299	0.060	0.038	0.014

CHLUMĚTÍN

Possible causes of failure of an aerated stabilisation pond are: toxic entry that would affect the activity of the microorganisms, equipment failure causing insufficient aeration, atmospheric conditions, such as low temperatures that affect the efficiency of the pond, and design problems, for example hydraulic short-circuiting causing short hydraulic retention times.

Probability that the aerated pond will fail to produce effluent of sufficient quality was calculated as:

$$P(N) = P_1 + P_2 + P_3 + P_4 = 0.015 + 0.015 + 0.01 + 0.005 = 0.045, \quad (6.11)$$

the stabilisation pond is thus 95.5% reliable. Monte Carlo simulation with 10 000 random situations predicted 765 failures, reliability is thus 92.35%.

Table 39. Chlumětín - possible causes of failure of a stabilisation pond and respective probabilities, $P(N_0)$ and IFs

cause of failure		probability [%]	failures when omitting P_n	$P(N_0)$	IF [%]
P_1	toxic entry	1.5	303	0.0303	34.70
P_2	equipment failure	1.5	319	0.0319	31.25
P_3	atmospheric conditions	1	363	0.0363	21.77
P_4	design problem	0.5	418	0.0418	9.91

Following table compares pollution discharged during the time when aerated pond does not comply with the concentration that has been established as sign of failure to pollution discharged from non-functional decentralised systems.

Table 40. Chlumětín - pollution discharged to the environment during WWTP failure

CHLUMĚTÍN		PARAMETER	BOD_5	COD_{Cr}	TSS	TN	N-NH_4	TP
pollution generation	load	[kg/day]	12.2	24.4	11.2	2.2	1.4	0.5
	concentr.	[mg/l]	533.3	1067	488.9	97.8	62.2	22.2
emission standards	p	[mg/l]	150	40	50	-	-	-
	m	[mg/l]	220	80	80	-	-	-
c- failure threshold	c_{fail} = (p+m)/2	[mg/l]	185	60	65	-	-	-
average c during failure		[mg/l]	229.1	500.1	216.5	-	-	-
aerated stabilisation pond	volume	[m^3/day]	1.0	1.0	1.0	1.0	1.0	1.0
	load	[kg/day]	0.232	0.506	0.219	-	-	-
small-scale WWTPs	5% systems	[kg/day]	0.609	1.218	0.558	0.112	0.071	0.025
	1.5% systems	[kg/day]	0.183	0.365	0.167	0.033	0.021	0.008

Aerated stabilisation pond is 92.35% reliable, which means that 7.65% of effluent has concentration above c_{fail}. For Bořice, this equals to 1 cubic meter that is only partially treated or untreated. The average concentration of untreated samples was determined using Monte Carlo simulation. Daily BOD_5 loads to receiving waters from aerated stabilisation pond during failures is 0.23 kg/day, which

equals to 85 kg/year. Decentralised system without central monitoring would discharge additional 222 kg BOD_5/year, assuming that 5% of the on-site system would discharge untreated wastewater.

If decentralised system would be centrally monitored, annual BOD_5 loads to receiving environment would be reduced to 67 kilograms of BOD_5, making centrally monitored decentralised system the most reliable option for Chlumětín.

6.3.1 Operational reliability assessment summary

Traditional centralised wastewater management is usually operated and supervised by expert operators with knowledge of treatment processes. Decentralised management is generally less controlled in terms of effluent quality, which raises concern about reliability of these systems. The aim of this scenario was to compare centralised and decentralised systems in terms of pollution that is discharged during non-standard and emergency situations.

Operational reliability assessment compared the ability of centralised and decentralised systems to fulfil their function, e.g. to produce effluent of sufficient quality. There is not much data yet on reliability and failures of small-scale WWTPs. Based on expert elicitation of a person authorised to carry out technical checks of on-site wastewater treatment systems [68], approximately 1-2% of decentralised systems do not work properly when the system consisting of multiple decentralised WWTPs is centrally monitored.

Reliability of constructed wetland and series of aerated stabilisation ponds was calculated from probabilities of individual reasons for failure to comply with effluent quality requirements. For mechanical-biological municipal WWTP Hrochův Týnec, FTA analysis was used to calculate reliability due to higher complexity of events that can contribute to system failure.

Calculated reliabilities were then re-calculated using the Monte Carlo simulation. Influence of individual reasons for failure on system failure was investigated.

The results showed that decentralised systems without any monitoring are the least reliable option, and would result in large pollution loads discharged into the receiving waters. However, when decentralised system of multiple on-site WWTPs is combined with centralised monitoring system, the risk of undetected failures is reduced to high extent. For all three case studies, centrally monitored decentralised system seemed to be the most reliable option, as the pollution loads to receiving waters were even lower than pollution loads discharged from centralised WWTPs during non-standard operational situations.

7 DISCUSSION

With increasing environment protection measures and requirements, greater emphasis is being placed on wastewater treatment in smaller agglomerations. However, lower population density in rural areas and dispersed settlements represent a challenge from engineering point of view, as traditional centralised wastewater schemes benefit from economies of scale and require large investment costs for construction of sewage network and centralised plant. Even when the construction costs of a municipal WWTP can be covered by government subsidies, for small municipalities operation, maintenance and rehabilitation costs of a centralised system can be a problem from financial point of view.

In addition, sustainability of centralised systems has been questioned recently due to fast transport of produced wastewater, sometimes together with stormwater, to centralised WWTP and consequent discharges of large volumes of treated wastewater which can have negative impact on the receiving waterbody. Moreover, combined systems represent environmental burden due to CSO discharges during heavy storms. Decentralised systems have been introduced as a viable alternative to the traditional scheme. In decentralised systems, wastewater is treated and discharged close to where it is produced, which eliminates the need for large transport networks, and fragments discharged pollution loads into multiple point sources, instead of one significant discharge from a centralised WWTP. The receiving environment is more likely to cope with smaller point sources, e.g. due to assimilative capacity of rivers, which represents another advantage of decentralisation. Stormwater, instead of being rapidly transported downstream, is accumulated or allowed to infiltrate, which helps maintain the water cycle or restore it to its natural regime.

However, the decentralised concept is still rather new and as such raises many concerns, mainly about environmental risks. The aim of this thesis was to provide a clear comparison of impacts on the environment of centralised and decentralised systems, in terms of pollution discharged to the receiving waterbody, energy required for operation, and system reliability. For the comparison, three case studies were selected.

While centralised systems in Tisovec and Chlumětín discharged less pollution when legislative requirements for discharges into surface waters were considered, decentralised systems produced significantly less pollution based on the real-life data. The presumption that small-scale WWTPs produce effluent of lower quality as compared to centralised WWTPs has been thus put into question. Small-scale WWTPs available on the Czech market discharge effluent with lower pollution

concentrations than centralised WWTPs, as suggested in declared efficiency scenario, although the legislative requirements for small-scale systems are still milder.

In some instances, pollution loads cannot be compared solely on quantitative basis, as shown by the case of municipality Bořice. Since there is no receiving surface body, decentralised systems would have to discharge their effluent into groundwater. Based on the anticipated effluent values from centralised and decentralised systems, it was determined that daily pollution load would be slightly lower if decentralised option was chosen. However, bodies of groundwater are more vulnerable to pollution, and therefore it is possible that the same pollution load which would not significantly affect a surface waterbody could have far-reaching impact on groundwater environment. In cases like this, detailed assessment of possible environmental risks arising from discharges into groundwater would be beneficial.

Energy assessment compared energy requirements for centralised and decentralised systems. It was confirmed that centralised systems consume less energy per volume unit treated due to economies of scale. However, for less densely populated areas, financial and energetic requirements for construction of sewage network can be enormous. Thus, the advantage of decentralised systems does not lie in lower energy consumption per volume unit treated, but in removing the need for construction of a sewage network. In addition, decentralised systems with zero energy requirements, e.g. septic tanks followed by soil filters, reach comparable treatment efficiencies as other decentralised systems, making them the optimal option for objects that are in use only during some parts of the year.

Criteria for controlling effluent quality differ significantly for municipal WWTP and small on-site systems in Czech Republic. While municipal WWTP are obliged to provide regular samples multiple times a year, on-site systems are checked only once every two years which gives raise to concerns about possible long-term environment pollution during undetected failures. In addition, centralised WWTPs are under supervision of expert operators, while decentralised systems are often negatively affected by the lack of knowledge and effort of the owner of the system. According to an experienced practitioner, approximately 5% of decentralised systems do not work properly and discharge untreated wastewater. However, with the implementation of a centralised monitoring system, percentage of dysfunctional systems can be reduced to 1-2%. Since monitored systems are by far more reliable than unmonitored WWTPs, the system of control could consider this factor and in the future, owners of unmonitored systems could be controlled more frequently to lower the chance of long-term dysfunction of a system. Introduction of obligatory monitoring of decentralised could be also considered, especially for protected or vulnerable areas.

8 CONCLUSIONS

The aim of this thesis was to compare centralised and decentralised wastewater and stormwater management in small municipalities based on environmental, energy, and operation related aspects. The results from the three case studies suggest that for municipalities Tisovec and Chlumětín, decentralised management with centralised monitoring systems would provide the most effective treatment of domestic wastewater. In Bořice, centralised option was considered more suitable due to absence of receiving surface waterbody and proximity to an existing WWTP.

Municipality Chlumětín is located in protected countryside area, where environment protection should be of highest concern. Proposed centrally monitored system of decentralised MBRs with treated wastewater and stormwater accumulation and reuse was selected as optimal due to very high quality of effluent and reliable operation. Although decentralised solutions, especially MBRs, have higher energy consumption as compared to centralised systems, their implementation in protected areas is justifiable due to high effluent quality and water reuse possibilities.

Proposed decentralised management in Tisovec is a series of centrally monitored SBRs. The thesis demonstrated that decentralised technologies can discharge smaller loads of pollution as compared to proposed constructed wetland. Due to approximately 1-km distances between 5 local parts of Tisovec, construction of a sewage network would require significant funding, therefore decentralisation is beneficial from both environmental and economic point of view.

Centralised wastewater treatment is considered preferable for municipality Bořice. Although anticipated pollution loads discharged to the environment from the existing WWTP in Hrochův Týnec are higher than pollution loads from investigated decentralised systems, the percental difference between loads from centralised and decentralised system is not as high as for Tisovec and Chlumětín. The rationale for selecting centralised option was the absence of a suitable receiving waterbody in Bořice. Groundwater discharges could in turn cause more far-reaching environmental impacts than discharging slightly higher pollution loads in the river Novohradka.

In addition, existing WWTP Hrochův Týnec has sufficient capacity to allow connection of Bořice without the need to increase plant capacity. Existing septic tanks and cesspits will be used for accumulation of stormwater which can be reused for irrigation during vegetation period to reduce consumption of potable water for purposes where water of non-potable quality can be used.

These findings are in agreement with results of thesis ‘Analysis of optimal degree of decentralisation of small municipalities drainage and treatment systems’ [33] that investigated

centralised and decentralised options in Tisovec, Chlumětín and Bořice from economic point of view. The thesis suggests that decentralised management in municipalities Tisovec and Chlumětín has significantly lower investment and operational costs. From economic point of view, centralised management was considered only in Bořice. Investment costs were still about three times higher than for decentralised management, but operational costs were slightly lower, and the absence of suitable waterbody for effluent discharges was also considered as an important factor.

In conclusion, decentralised systems have proven their ability of producing effluent of high quality. For all three cases studies it was shown that when considering declared commonly achieved values, decentralised systems produce lower daily load to receiving environment, and advanced systems such as MBRs are capable of providing effluent that is suitable for reuse as water for non-potable purposes. Decentralised systems still have higher energy requirements as compared to centralised systems, but on the other hand they do not require significant amounts of resources and energy for construction of large sewage networks, and previous research has shown that for small municipalities, decentralised systems are more favourable also from economic point of view.

Selection of optimal technology and centralisation rate of wastewater and stormwater management will always depend on local conditions. This thesis demonstrated that as far as environmental impacts are concerned, modern decentralised systems represent a viable alternative to traditional centralised approach, and decentralised wastewater and stormwater management in small agglomerations has the potential to significantly reduce the environmental impacts of human activity.

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