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VALENTINA RUTAR POLANEC

RAZVOJ ZELENE STENE ZA ČIŠČENJE SIVE VODE IN PRENOS TOPLOTE

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Povzetek

Komercialno dostopne zelene stene nudijo številne prednosti v urbanem okolju. Kljub temu, v primeru namakanja s pitno vodo, lahko postanejo velike porabnice vode in zatorej zelene stene niso primerne za uporabo v suhem podnebju. Če bi bile zasnovane za čiščenje sive vode (SV), bi postale ekonomične in primerne za vsa okolja, zlasti v sušnih območjih. SV je obetaven vir vode za namakanje, predvsem zaradi njene stalne proizvodnje ter vsebnosti organskih in hranilnih snovi za uspevanje rastlin. Poleg tega, se SV lahko uporablja kot vir toplote za predhodno segrevanje sanitarne tople vode. V okviru magistrskega dela smo na podlagi pregleda strokovne literature predlagali najprimernejši sistem za prenos toplote v kombinaciji z zelenimi stenami za namen nadaljnjih študij o SV. Nato smo zasnovali pilotni sistem zelene stene za čiščenje SV ter v razmeroma kratkem času, tj. štirih mesecih, za vzpostavitev primernih bioloških pogojev, izvedli laboratorijsko-pilotski poskus. Cilji magistrskega dela so bili doseči priporočila koncentracij snovi na iztoku iz zelene stene ter določiti dizajn parametre zelenih sten za čiščenje SV z vodoravnim tokom. S čiščenjem SV na zeleni steni smo dosegli 70 % KPK, 74 % BPK, 20 % NH_4-N in 72 % NO_3-N učinkovitost, z organsko obremenitvijo SV 33 g KPK/m^{2}d in 7 g BPK/m^{2}d. Ob koncu eksperimenta so koncentracije snovi na iztoku iz zelene stene ustrezale več različnim nacionalnim smernicam za namakanje, izpust v okolje in ponovno uporabo.

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Abstract

Commercially available green walls can be beneficial to urban landscapes, but if designed in a way such that tap water is used for their irrigation, they can be quite voracious consumers of water that are not optimal for use in arid areas. If they were designed to treat greywater (GW), they could become economical and suitable for all environments, especially in dry climates. GW is a promising irrigation candidate given its continuous production and its composition of organic matter and nutrients to support healthy plant growth. In addition, GW can be used as heat source for preheating domestic hot water. In this thesis, heat recovery systems were overviewed and those that are most suitable were proposed for further studies on GW in combination with green walls, while a green wall system for GW treatment was tested in detailed experiment by designing and building a pilot. Over a relatively short period of time, i.e. four months, for the purpose of establishing suitable biological conditions, a laboratory-pilot-scale experiment was conducted. The objectives were to provide recommendations and determine the design parameters for GW treating green wall systems with horizontal flow. The treatment performance of the green wall reached a removal of up to 70% for COD, 74% for BOD, 20% NH_4-N and 72% NO_3-N, with an organic loading rate of 33 g COD/m^2d and 7 g BOD/m^2d. The effluent from the green wall design satisfied several national guidelines for irrigation, environmental and restricted reuse.

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A LIST OF ABBREVIATIONS

Abbreviation	Unit	Definition
BOD	[mg/L]	Biochemical oxygen demand
COD	[mg/L]	Chemical oxygen demand
NH_3	[mg/L]	Ammonia
NH_4-N	[mg/L]	Ammonium
NO_2-N	[mg/L]	Nitrite
NO_3-N	[mg/L]	Nitrate
PO_4-P	[mg/L]	Phosphate
TSS	[mg/L]	Total suspended solids
DO	[mg/L]	Dissolved oxygen
Ksat	[m/s]	Hydraulic conductivity
EC	[$\mu S\ cm^{-1}$]	Electrical conductivity
Redox	[mV]	Reduction oxidation potential
OLR	[g/m^2d]	Organic loading rate
OL	[mg/L]	Organic load
HRT	[h]	Hydraulic retention time
HLR	[m/s]	Hydraulic loading rate
HL	[m^3/L]	Hydraulic load
GW	/	Greywater
WW	/	Wastewater
NBS	/	Nature-based solutions
DHW	/	Domestic hot water
DWHRS	/	Drain water heat recovery system

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1 INTRODUCTION

What importance does water recycling have when there is plenty of it on Earth? About 71% of the Earth's surface is covered with 1400 million cubic km of water, and the oceans hold about 96.5% of all the Earth's water. However, of that total amount of fresh water stored in the ground, rivers, lakes and accessible for human use is less than 0.77% and less than 0.003% is appropriate for human consumption (Shiklomanov, 1993). In addition, the distribution of rainfall and consequently its availability is highly uneven across the planet. Nevertheless, the amount of water on earth is kept largely the same due to the hydrological cycle.

What is changing is the availability and quality of clean water, which is mainly affected by rapid population growth, human activity and needs. This causes pressure on the natural environment, reduces resilience, and limits ecosystem services (UNESCO, 2015, Strungaru *et al.*, 2015, Gaiser *et al.*, 2008). In 2017, the global population was 7.6 billion people, a number that is projected to increase by 29% over the next three decades, resulting in continuously increasing water demand (United Nations Department of Economic and Social Affairs Population Division, 2017). In fact, world demand for freshwater will increase 55% between 2000 and 2050 (OECD, 2012). Roughly, the amount differs for each continent; however, of the world's freshwater 69% is used for agriculture, 19% for industry, and 12% for municipal use (FAO, AQUASTAT, 2020). The increased demand will mainly come from industry and manufacturing with a 400% increase, from electricity-generation with 140% increase and domestic freshwater use with a 130% increase (Marchal *et al.*, 2011).

Thus, the demand for utilizing new water sources is increasing. Solutions like purchasing water from other countries, exploiting more distant (surface water bodies) and deeper (groundwater) sources, constructing new dams and seawater desalination plants are clearly unsustainable. Therefore, not only reclaiming water from wastewater is needed in order to face the current water scarcity and pollution in our planet (Garnier *et al.,* 2015) but to rethinking the entire water infrastructure and water use habits.

During the past 50 years, the "urbanization and industrialisation" phenomena has become faster than ever, causing the occurrence of Urban Heat Islands (UHI) effect (Rizwan *et al.,* 2008). It is expected that heat waves in urban areas will last longer and occur more frequently (Ward *et al.,* 2016). Furthermore, in the last decade, the increasing demand, population growth, and the limited availability of fossil fuels have collectively led to an energy crisis (Coyle and Simmons, 2014), leaving researchers continuously in search for new sources of energy. The global primary energy demand is expected to increase at an annual rate of 1.3% until 2035, with the highest increase in India of 2.7% already today (International Energy Outlook, 2017). Tremendous effort has been invested in finding solutions to decrease the consumption of fossil fuels. Renewable energy sources such as hydropower, solar, biomass, wind, solid wastes geothermal (U S Environmental Protection Agency, 2019), and waste heat from air conditioning (Abd El-Baky and Mohamed, 2007) are being explored by many countries (US Energy Information Administration, 2019).

Here, too, suitable water management can play a crucial role. Wastewater-based heat recovery (Frijns *et al.*, 2013) is being considered as an alternative energy source (Ravichandran *et al.*, 2002). Depending on wastewater flow and temperature, heat exchangers and heat pumps (or a combination of both) can be used for extracting heat energy from wastewater (Arnell *et al.*, 2017) and consequently, saving energy for residential water heating which accounts for 4–6% of the total national energy demand. In addition, a range of nature-based solutions (NBS) (Langergraber *et al.*, 2020) for wastewater treatment, e.g. treatment wetlands, green walls, green roofs and more, offer multiple benefits when designed for the urban environment: apart from wastewater treatment they mitigate UHI, reduce energy consumption of buildings, provide biodiversity and amenity. Thus, in order face the issues listed above water management and technologies that can treat and reuse wastewater as an alternative resource of water and energy, are becoming significantly important (Ghaitidak and Yadav, 2013; Li *et al.,* 2009; WWDR, 2015).

To design, construct, operate and maintain a building's energy, new materials and water are utilized, and certain amounts of waste are generated, adversely affecting the environment and people's health. In order to limit these effects "green building systems or living architecture" must be introduced. Living architecture focuses on integrating ecological functions into the buildings to catch, store, and filter water, purify air, and process other nutrients. Moreover, it promotes biophilia, the documented health benefits associated with being in touch with living systems in the built environment (Susan, 2008).

Similarly to treatment wetlands, ideas for potential alternative wastewater treatment systems exist, that embodies the philosophy of green architecture and is recognised under different terms such as green walls, wet walls, living walls, wet facades, etc. (Medl, Stangl and Florineth, 2017), which has not been utilised sufficiently until now. "Green wall" will be the main common term used for this technology, since the terminology is still being specified among researchers and is also discussed later in this thesis.

The main objectives of this master's thesis were: (1) a broad overview of the literature on greywater and green walls, (2) designing the system, (3) operating it for a certain period of time, and (4) comparing the results with the previously researched systems. In addition, the objectives were to collect the existing design parameters and alter them for a specific, linear green wall system with planter boxes (beds) and horizontal water flow. In Section 2 of this thesis, various approaches to wastewater and its fractions (greywater, blackwater) are introduced. Then in Section 3, 4, and later in section 6, the reuse of greywater as a potential source of water, nutrients, and energy is further discussed and described, along with greywater characterisation and its treatment technologies. In Section 5, heat recovery systems that recuperate energy from greywater are investigated and the ones most suitable proposed for further studies on greywater in combination with green walls. Furthermore, in Section 7, the history of green wall technology is overviewed through a survey of the scientific literature, and the design parameters from various previous studies are collected and compared. In Section 8, the design, installation, and experiment procedure on a green wall greywater treatment are described in detail. Lastly, Section 9 contains a presentation of results and a discussion.

The pilot green wall greywater treatment system with heat exchanger was designed and built in a lobby of the Faculty of Civil and Geodetic Engineering in Ljubljana in collaboration with the Faculty of Mechanical Engineering. In an experimental period of four months the system was tested for greywater treatment efficiency. The experiment was divided in three phases that describe three different setups. In Phase I., the greywater used was heavily loaded; no aeration was used and no vegetation was planted. In Phase II. the greywater load was halved and aeration was introduced, and in Phase III. vegetation was planted. Chemical and physical parameters were monitored throughout the experiment. After the experiment was over, further improvements on the design were proposed.

2 APPROACHES TO WASTEWATER MANAGEMENT

In this chapter, the focus is on domestic wastewater generated from different uses in households. Conventional wastewater management will be described for a better further understanding of the main topic which is greywater treatment, reuse and the importance of it in the future. Data about the European wastewater treatment regulations and the successfulness of their application and its current conditions in Slovenia will be presented.

2.1 Domestic wastewater characterisation

Domestic/household wastewater is part of the urban water that is generated by (1) drinking and metabolizing water by humans and (2) different household uses like toilet flushing, cleaning, laundry, shower. Household wastewater is thus mainly composed of these two fractions, called blackwater and greywater. Blackwater consists of the discharges from toilets and it can be further divided into urine and faeces (see Figure 2). Especially when collected with vacuum toilets, blackwater contains and phosphorous nitrogen in high concentrations and most of the pathogens, hormones and pharmaceutical residues (Zeeman *et al.* 2008). Greywater is defined as urban wastewater without any input from toilets and so generally includes sources from baths, showers, hand basins, washing machines, dishwashers and kitchen sinks (Jefferson *et al.*, 2004).

Total wastewater (grey and blackwater) production strongly depends on domestic use of fresh water of an average person which is different around the globe, e.g. 128 L/PE/d in Europe (EurEau, 2017), 104 L/PE/d in Slovenia (SiStat, 2020), 123 L/PE/d in Germany (Statistisches Bundesamt, 2020), 150 L/PE/d in England (Ghaitidak and Yadav, 2013) and up to 314 L/PE/d in America (Cheryl et al, 2015). Across the developing countries there is a strong variation in the water available for users, as developed cities, for example in India, have an average water supply of 80 L/PE/d (Tamil Nadu) to 540 L/PE/d (Chandigarh) (CPCB 2009). Standard terms are used to identify and understand the impact of individual pollutants in wastewater on the environment. The most convenient way to measure pollution is by the impact of an adult person, i.e. one resident of 1 P or in professional terminology 1 PE, which stands for population equivalent or unit per capita loading in one day. The pollution from industry or agriculture can be calculated according to the values caused by one inhabitant and therefore the values are presented as population equivalents. Typical load per adult in one day is seen in Table 1.

Table 1: Specific loads per PE a day in g/(PE*day) (ATV-DVWK-A 131E)

	Parameter	Load per 1PE	Unit
Biological oxygen demand	BOD	60	g O_2/(PE*day)
Chemical oxygen demand	COD	120	g O2/(PE*day)
Suspended solids	TSS	70	g /(PE*day)
Total Kjeldahl nitrogen	TKN	11	g /(PE*day)
Ammonium nitrate	NH_4-N	75	%
Organic nitrogen	N_{org}	25	%
Total phosphorus	P	1.8	g /(PE*day)

2.2 Conventional (centralized) wastewater management

This approach of wastewater management includes collection of total generated household wastewater - many times mixed also with storm water - in sewer systems that convey the water to a central wastewater treatment plant (WWTP). At the WWTP the wastewater is treated and discharged to the receiving waters. Depending on the types of contaminants removed, wastewater treatment is divided into various levels, i.e. preliminary, primary, secondary, tertiary and advanced. According to the techniques used for removal of contaminants, treatment methods can be classified as physicochemical and biological methods. Physicochemical methods include grit removal, screening, sedimentation, coagulation, flocculation, sludge thickener, multimedia filtration, ion-exchange, adsorption, reverse osmosis and ultrafiltration. Biological methods are classified as aerobic and anaerobic. Aerobic methods are further divided as suspended growth (activated sludge process, aerated lagoon, waste stabilization pond, etc.) and attached growth (tricking filter, rotating biodiscs, fixed film reactors, treatment wetlands, etc.). Anaerobic treatments comprise contact beds, sludge digesters, up-flow anaerobic sludge blanket reactorsand, anaerobic ponds and lagoons (Li *et al.,* 2009; Arceivala and Asolekar 2007).

2.3 Challenges of conventional (centralised) wastewater systems

2.3.1 High costs in dispersedly populated areas

Conventional centralised sewage system and most of the treatment technologies have a high energy demand and high construction, operation and maintenance costs (Park *et al.,* 2015; Gagnon *et al.,* 2010), this especially applies to the countries where population is mostly dispersed and agglomerations are small. In Europe, according to directive 91/271/EEC, wastewater regulations in agglomerations are only discussed when population equivalent is higher than 2.000 PE and below 2.000 PE only when wastewaters are discharged into the sewage system. Wastewater from agglomerations below 2.000 PE and from areas outside of agglomerations are discussed in the directive 2008/98/EC. According to the Government of Republic of Slovenia the estimated costs of construction of public sewer network or a municipal wastewater treatment is higher in less populated areas than in densely agglomerated areas:

- The investment in a public sewer network in agglomerations with a total load exceeding 2.000 PE is 1.400 EUR/PE, whereas the investment in a public sewer network in agglomerations with a total load of less than 2.000 PE is 2.000 EUR/PE,
- The investment in a municipal WWTP in agglomerations with a total load exceeding 2.000 PE is 400 EUR/PE, whereas the investment in a municipal WWTP of agglomerations with a total load of less than 2.000 PE is 600 EUR/PE (Government of RS, 2019).

2.4 Decentralized wastewater management

2.4.1 Small agglomerations and dispersed settlement pattern

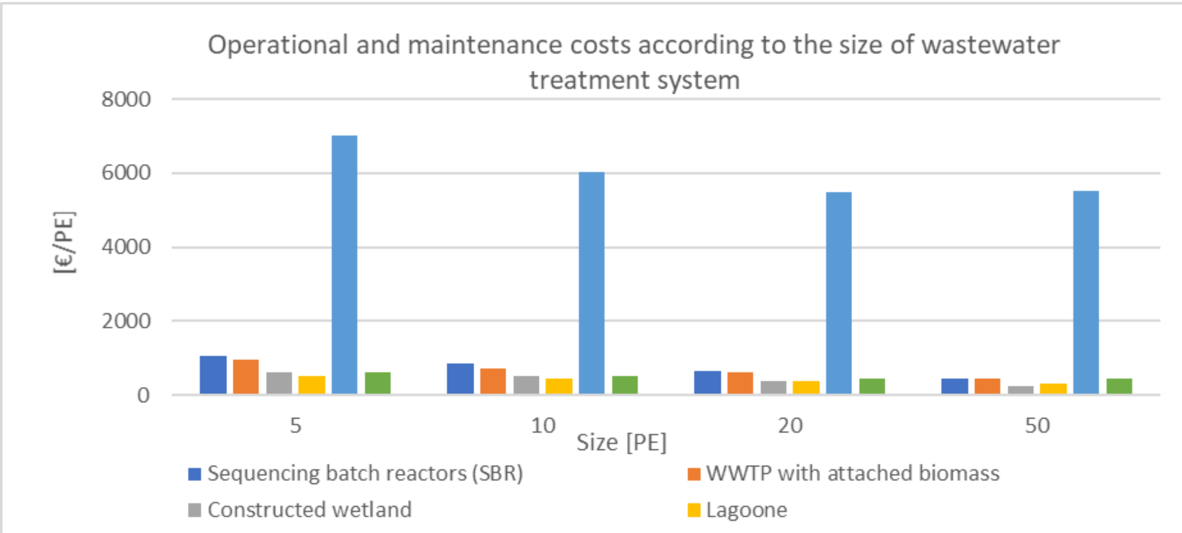
A dispersed settlement is the scattered pattern of households in a particular area. This form of settlement is common in the world's rural regions. Slovenia's settlement is distinctly dispersed, and its settlements are historically unequally distributed, while in the decades following World War II, under the influence of socio-economic change, the settlement non-uniformity increased even further. On behalf of the investments listed above, the cost of construction of sewage systems and municipal wastewater treatment plants are high in Slovenia due to the small agglomerations and its dispersed settlement pattern. Typically, rural areas make up 30.5% of the total territory of the country in which 38.5% of the total population is living (RS, MAFF, 2006). To illustrate, in Slovenia with total load of 2.055.003 PE, there are 1535 of agglomerations of which the majority, that is 91%, have less than total load of 2.000 PE and 75% have a total load of only 50–500 PE (Government of RS, 2019).

In addition, 22% of total Slovenian load do not belong to defined agglomeration areas (lower than 50 PE units) which means that 417.475 PE need to be treated with small on-site wastewater treatment units i.e. small biological wastewater treatment plants. Current valid wastewater management for individual houses are sealed septic tanks, one of the most expensive solutions due to the needed too frequent cleaning and transporting the waste content to the nearest WWTP. As presented in Graph 1, sealed septic tanks demand highest operational and maintenance cost comparing to other wastewater treatment systems i.e. SBR, lagoons, WWTP with attached biomass, treatment wetlands and unsealed septic tanks (Kompare *et al.*, 2007).

The construction and costs of small WWTP in places out of the reach of public services (out of defined agglomerations) have to be carried out and covered by people alone, latest at the time of reconstruction of their houses, which usually becomes the case of disinterest, since people see small WWTP as an unnecessary expense.

To conclude, in places where dispersed settlement pattern prevails as in Slovenia, it is reasonable that municipalities invest more into decentralised wastewater system with more small WWTP such as treatment wetlands and get familiar with some other green technologies on the market and where enough land area is available, rather than investing in construction of centralised sewage network system and municipal WWTP. It is also of great importance to start considering wastewater as a source of resources and to include this fact when calculating the costs, to become wastewater management in general more sustainable, which is also the aim of this thesis. Therefore, a different approach to centralized, i.e. decentralized management is more favourable in Slovenia.

However, these policies and regulations again do not include agglomerations specified below the 50 PE. In fact, 22% of total Slovenian load is out of agglomeration areas which means that 417.475 PE yet needs to be treated with small on-site wastewater treatment units i.e. small biological wastewater treatment plants or treatment wetlands. Moreover, according to "Uredba o odvajanju in čiščenju komunalne odpadne vode (Uradni list RS, št. 98/15, 76/17 in 81/19)" of Slovenia linked to the directive 2008/98/EC of European union, the construction and costs of small WWTP have to be carried out and covered by people alone, latest at the time of reconstruction of their houses, which usually becomes the case of disinterest, since people see small WWTP as an unnecessary expense. Therefore, by letting it stay as it is, municipalities will still have to cover for high maintenance costs due to their obligation of regularly cleaning the sealed septic tanks (Kompare *et al.*, 2007). As presented in Graph 1, sealed septic tanks demand highest operational and maintenance cost comparing to other wastewater treatment systems i.e. SBR, lagoons, WWTP with attached biomass, treatment wetlands and unsealed septic tanks.



Graph 1: Operational and maintenance costs (€) according to the size of wastewater treatment system (PE) (Kompare *et al.*, 2007)

This scenario suggests that it is still a long way to find an economical solution for small WWTP which would also include the green wall system technology presented in this thesis.

2.5 Source separation approach

Source separation approach is one step further to decentralization. It collects waste fluxes (greywater, and blackwater – jointly or urine and faeces separately) in separate systems (pipes) with the aim to facilitate the treatment and the recovery of resources from wastewater. In source separation systems thus, waste fluxes are separated according to their content of recoverable compound. Domestic wastewater can be separated by collecting greywater (water from showers, laundry, hand washing basins and kitchen) and blackwater (flushed water from toilets) in separate piping. Further, by using low or zero-flush toilets, blackwater can be separated to urine and faeces (Masi *et al.*, 2020). Greywater recycling represents a plausible approach to achieve greater water sustainability and resiliency, if treated to nonpotable instead of potable standards, less resource-intensive treatment processes may be used, combined with reducing discharge to wastewater treatment plants and offsetting potable water demand (Ma *et al.,* 2015; Xue *et al.,* 2015). In other words, separating the greywater and blackwater and treating greywater locally, would reduce a lot of volumetric burden on the existing centralized conveyance and treatment system. By separating these two a less diluted wastewater can be achieved that is easier to treat. Greywater is less polluted compared to the blackwater so it can be treated efficiently by separating it from the source of origin. Use of this approachproves to be economical for both conveyance and treatment. Wastewater reuse increases the total available water supply (Noah 2002; United Nations Environment Programme 2005). Moreover, the reuse of greywater lowers the total costs for wastewater handling, as the load of water being processed in the treatment plants reduces (Eriksson *et al.*, 2002). Especially, this goes hand in hand with tourist cities during the dry or tourist season, when greywater reuse can be applied to hotels, where the water demand is highest or in any other densely populated places that have large or high buildings such as airports, shopping malls, skyscrapers, office buildings, casinos, faculties, prisons, dormitories, monasteries, gyms and similar (Masi *et al.*, 2016).

3 POTENTIAL OF GREYWATER REUSE

Globally, traditional consumption patterns are still mostly based on the ‘take-make-dispose’ concept, where there is no planning for resources reuse or regeneration of the natural systems (Ellen MacArthur Foundation, 2013). A better approach, slowly coming into practice, is circular economy (CE). It involves a regenerative industrial system where resource input and waste, as well as any kind of leakage, are minimised by slowing down and closing material and energy loops (Geissdoerfer *et al.*, 2015; Ghisellini *et al.*, 2016; Kirchherr *et al.*, 2017). Kirchherr *et al.* (2017) defined a CE as a system based on business models promoting reducing, reusing, recycling and recovering materials operating at different levels, with the aim of achieving sustainable development. In this chapter advantages, risks and legislation according to greywater reuse will be discussed, since it presents a great example of circular economy paradigm.

3.1 Advantages of greywater reuse

3.1.1 Environmental benefits

The separation of household wastewater streams provides advantages for wastewater management allowing for resource recovery and closing the loop of nutrient cycling (Otterpohl, 2002). Separating greywater from household wastewater has many advantages. Greywater has the potential to carry less organics, nutrients and pathogens than municipal wastewater in the absence of faeces, urine and toilet paper and is therefore thought to be easier to treat for reuse purposes (Abu Ghunmi *et al.,* 2011; Eriksson *et al.,* 2002; Ghaitidak and Yadav, 2013; Li *et al.,* 2009; Mayer *et al.,* 1999) in non-potable water applications such as infiltration, irrigation, toilet flushing, washing water, etc. (Hernández Leal *et al.,* 2011). Greywater recycling has been proved to be efficient in reducing water demand and lower the total costs for wastewater handling, since there will be a reduced load of water to the treatment plants (Eriksson *et al.,* 2002). A study from Friedler and Galil showed that in the year 2023 with greywater reuse systems units installed in 20–30% of all houses, reuse of greywater in the urban sector in Israel with projected population of 10 million people, could save up to 45 million m^3/year in toilet flushing and up to 10 million m^3/year in garden irrigation. This amounts to about 5% of the total future urban water demand in the country and equals the capacity of a medium size seawater desalination plant. Griggs *et al.* (1998) identified greywater reuse for irrigation, urinal and toilet flushing as major water conservation measures, since the water that is used for toilet flushing in many countries today is of drinking water quality (Karpiscak, Foster, and Schmidt, 1990). Toilet flushing can reduce the in-house net water consumption by 40–60 L/PE/d that is 30% of the total household water consumption, leading to 10–20% reduction of the urban water consumption, which is significant especially under water scarcity situation (Boyjoo, 2013). Greywater reuse can be most beneficial in urban areas where integrated water management is needed. This has been practiced in many countries; including USA, Australia, Japan, Sweden, Germany, UK, and Canada (Nolde, 1995; Waller *et al.,* 1996; Mustow *et al.,* 1997; Fittschen and Niemczynowics, 1997). For example, in California, Young and Holiman (1990) identified 380 housing schemes directly reusing greywater for non-potable uses. Similarly, there are 840 in building recycling units, 42 district systems and 27 municipality-based schemes providing reclaimed water in Japan (Aya, 1994). Additionally, many office buildings, apartment blocks and municipal buildings incorporate wastewater reuse for toilet flushing in the city of Tokyo (Asano *et al.,* 1996). In Germany, greywater from bathrooms has been successfully reused. It has been shown that it is technically feasible and health requirements can be met. Substantial volumes of water 15–55 L/PE/d can be reused (Nolde, 1999). Furthermore, outdoor greywater reuse can be applied for irrigation of lawns on athletic fields, college campuses, cemeteries, parks and golf courses, agriculture areas or domestic garden which consume a considerable amount of water in some semi-arid regions such as Australia, California, Israel, etc. and seasonal tourist places (Boyjoo *et al.,* 2013). It can be used for maintenance of roads, pavements and bicycle lanes cleanliness (Okun, 1997). Washing of windows and vehicles, boiler feedwater, concrete production and fire protection are examples of other suggested usages (Okun, 1997; Santala *et al.,* 1998).

3.1.2 Economic benefits of recycling system

There have been many reports on greywater reuse resulting in significant savings in terms of water cost and demand for fresh water. For example, Sheikh (1993) reported that when greywater effluents were utilized for irrigation water in the city of Los Angeles, 12–65% of water savings were observed. Alike, Jeppesen (1996) reported that reuse of greywater for lawn gardening and toilet flushing in Australia could achieve water savings from 30 to 50% of total water usage by a household. Edwards and Martin (1995) reported that demand for toilet flushing can be satisfied by reusing water from showers, baths and laundry. Moreover, water supplied by wash basins alone can result in 20 to 30% reduction in the water demand for toilet flushing (Dixon *et al.,* 1999). Since potential greywater demand within the urban environment is significantly lower than its production, it is possible not to recycle all greywater streams, but rather to reuse the less polluted ones (Friedler, 2004). Light greywater (wastewater discharged from showers, baths and washing basins) represents an attractive alternative water source for non-potable uses. It requires minimum treatment which increases its suitability for on-site treatment and reuse (Fowdar *et al.*, 2017, Eriksson *et al.,* 2002). A reduction of saving 9–46% of potable water, can be achieved by reusing light greywater for other water consuming activities (Boyjoo *et al.,* 2013).

3.2 Risks and legislation

3.2.1 Risks

As medical and health professionals view blackwater as the most significant source of human pathogens, separation of greywater from blackwater will reduce the danger posed by such pathogens as greywater does not contain faeces. The organic content typical of greywater decomposes much faster than the content typical of blackwater, indicating that the decomposing matter in blackwater will continue to consume oxygen for longer; further away from the point of discharge (NSW Health, 2000). What makes greywater ideal for reuse is its reliability due it is a constant water source, contains nutrients and has low concentration of pathogens compared to mixed wastewater and blackwater (Eriksson *et al.,* 2002). However, greywater nutrient concentrations and organic loads come in great variations which is strongly dependent on individual user's behaviour (Imhof and Mühlemann, 2005). It should be taken into consideration that the greywater is not always pathogen-free. Both inhaling and hand to mouth contact can be dangerous. According to the Department of Health Western Australia (2010) it may contain high levels of pathogens (bacteria, viruses, protozoa, helminths), normally found in mixed wastewater. This is particularly true in hospitals and households where a resident is sick or with infants where diapers are routinely laundered. Therefore, greywater may exhibit negative aesthetic and environmental effects and pose health risks, especially in warm climates where higher ambient temperatures increase organic matter degradation and enhance pathogens regrowth (Eriksson *et al.,* 2002), hence a number of problems related to the reuse of untreated greywater may occur. Therefore, risk of disease spreading, due to exposure to microorganisms in the water, will be a crucial point if the water is to be reused for e.g. toilet flushing or irrigation, due to spreading the microorganisms present in the water in the form of aerosols that are generated as the toilets are flushed (Albrechtsen, 1998; Christova-Boal *et al.*, 1996; Feachem, 1983). Greywater may also contain high levels of organic matter, suspended solids, disinfectants, shampoos, detergents, bleaches, chemicals derived from soaps, dyes, toothpaste, mouthwash, caustic dishwashing powders and other products (UN Water, 2015).

The risk for pollution of soil and receiving waters due to the content of these different pollutants is another question that has been raised concerning infiltration and irrigation with greywater. For instance, Christova-Boal *et al.* (1996) stated that infiltration and irrigation may lead to elevated concentrations of detergents (for example) in the soil and some plants may suffer due the alkaline water. These pollutants, xenobiotic organic compounds (XOCs), originate from the chemical products (soaps, detergents, etc.) used in the households such as for personal care products and cleaning detergents. Many are synthetic and their effect and spreading is only partially known. Untreated greywater has potential for salt build up in certain soils. The soaps are alkali salts of long-chained fatty acids, while the detergents consist of surfactants as well as several other chemicals to improve the function e.g. builders, bleaches, enzymes, etc. Storage of greywater promotes rapid increase of microorganisms and lack of maintenance can lead to mosquito nuisance (Noah 2002; Arceivala and Asolekar 2007).

The greywater that is going to be reused must also be of satisfactory technical quality. Suspended solids may cause clogging of the distribution system. This often requires prior treatment to remove suspended solids and organic constituents in order to reduces the risk of clogging and guarantee a risk-free service of water for reuse applications other than potable water (Nolde, 1999). Another related problem is the risk of sulphide, which will give offensive odours and thereby cause public nuisance (Jeppesen, 1996). Nevertheless, according to the further utilization, greywater should respect hygienic safety and environmental tolerance (Eriksson *et al.,* 2002). Therefore, a level of caution must be exercised with greywater reuse. Overall, it is important to characterise the quantity and characteristics of domestic greywater when evaluating the possibilities for reuse, including the need for pre-treatment (Friedler, 2004; Karpiscak Foster and Schmidt, 1990).

3.2.2 Legislation

The need to minimise health and environmental risks of water reuse has led to the development of guidelines and regulations by some international and national organisations for the safe use of treated wastewater. From a reuse perspective greywater is considered as a wastewater, thus greywater reuse has to comply with wastewater reuse standards of existing legislation (Atanasova *et al.*, 2017). Globally, water reuse standards are variable and governed by the intended use of the treated effluent. In general, limits are imposed on specific parameters to reduce nuisance odours and algal growth as well as to protect environmental and human health (Li *et al.* 2009, Abu Ghunmi *et al.* 2011; Ghaitidak and Yadav, 2013). Such guidance is provided in the form of a risk management framework for the beneficial and sustainable management of water reuse systems. Examples include guidance provided by international organisations such as the World Health Organization (WHO), and national organisations of federal governments such as the US Environmental Protection Agency (USEPA) and, in Australia, the Natural Resource Management Ministerial Council, the Environment Protection and Heritage Council, and the Australian Health Ministers Conference (NRMMC-EPHC-AHMC). These guidelines can be used by states that have limited, or no, regulations or guidelines (Sanz *et al.*, 2014). Globally, a key reference for safe water reuse are the WHO's guidelines “Guidelines for the safe use of wastewater, excreta and greywater” with their last edition in 2006 (WHO, 2006). While the guidelines provide a framework for human safety in water reuse practices, they are not covering regulatory aspects (Atanasova, *et al.,* 2017). The WHO guidelines only refer to the safe use of wastewater in agriculture and aquaculture (WHO, 2006). International ISO standards on the reuse of reclaimed water were published in the years 2017, 2018 and 2019:

- ISO 20760-1:2018, Water reuse in urban areas — Guidelines for centralized water reuse system — Part 1: Design principle of a centralized water reuse system.
- ISO 20760-2:2017, Water reuse in urban areas — Guidelines for centralized water reuse system — Part 2: Management of a centralized water reuse system.
- ISO 20761:2018, Water reuse in urban areas — Guidelines for water reuse safety evaluation — Assessment parameters and methods.
- ISO/DIS 23056:2019, Water reuse in urban areas — Guidelines for decentralized/onsite water reuse system — Design principle of a decentralized/onsite system.

Next, at the national level USA’s USEPA issued recommended guidelines “Guidelines for Water Reuse” with the last update in 2012, though official standards have yet to be adopted (USEPA, 2012). The guidelines among other cover requirements for treated effluents from WWTP for urban reuse - restricted and unrestricted - including limit values of parameters like TSS, BOD, COD, Turbidity, Bacterial indicators and Pathogens (USEPA, 2012). California, a state that experiences chronic water supply crises and thus has a strong incentive to reuse water, is often recognized as having some of the most stringent state-level standards mostly due to the requirement for a 5 log reduction of poliovirus or similar virus (CDPH, 2010).

Outside of the US, Australian national guidelines for water reuse (NRMMC-EPHC-AHMC, 2006), advocate a risk management framework based on the WHO guidelines (WHO 2006) and also include limit values of pollutants (similarly as in USEPA, 2012) for different treatment processes and on-site controls for designated uses of recycled water. In contrast to WHO, USEPA and the Australian guidelines also consider several treated wastewater applications such as aquifer recharge and irrigation of golf courses.

At the EU level water reuse is encouraged in the Urban Wastewater Directive (91/271/EEC of 21 May 1991), where the level of reuse and development of appropriate standards are left to each member state.

Recently new regulation on minimum requirements for water reuse 2018/0169 (COD) has been issued, however it only applies to agricultural reuse.

Several countries and states around the world have/or are working on the local guidelines for reuse of treated wastewater for nonpotable reuse. Local standards for water reuse in Europe are most notably implemented in Cyprus, France, Italy, Greece, Portugal and Spain (Alcalde and Gawlik, 2014) and are listed in the Table 2.

Table 2: Most representative standards on water reuse from EU Member States (Sanz *et al.*, 2014)

Country	Standards reference	Issuing institution
Cyprus	Law 106 (l) 2002 Water and Soil pollution control and associated regulations KDP 772/2003, KDP 269/2005	Ministry of Agriculture, Natural resources and Environment Water development Department (Wastewater and reuse Division)
France	JORF num.0153, 4 July 2014 Order of 2014, related to the use of water from treated urban wastewater for irrigation of crops and green areas	Ministry of Public Health Ministry of Agriculture, Food and Fisheries Ministry of Ecology, Energy and Sustainability
Greece	CMD No 145116 Measures, limits and procedures for reuse of treated wastewater	Ministry of Environment, Energy and Climate Change
Italy	DM 185/2003 Technical measures for reuse of wastewater	Ministry of Environment Ministry of Agriculture Ministry of Public Health
Portugal	NP 4434 2005 Reuse of reclaimed urban water for irrigation	Portuguese Institute for Quality
Spain	RD 1620/2007 The legal framework for the reuse of treated wastewater	Ministry of Environment Ministry of Agriculture, Food and Fisheries Ministry of Health

The standards of Cyprus, France, Greece, Italy and Spain are included as regulations in the national legislation. In Portugal, the standards on water reuse are guidelines, but they are taken into consideration by the national government when issuing any water reuse permits in the country. All the standards evaluated refer to the reuse of urban and industrial wastewater effluents, except the standards of Cyprus and Portugal which refer only to urban wastewater. Most of the local standards for greywater reuse are intended not only for agricultural, but also for urban and industrial applications (Sanz *et al.*, 2014). The standards apply for each individual use of greywater such as listed and summarised in the Table 3.

Table 3: Intended uses for water reuse included in the standards of EU Member States (Sanz *et al.*, 2014).

Intended use of reclaimed water	**Cyprus**	**France**	**Greece**	**Italy**	**Portugal**	**Spain**
Irrigation of private gardens						√
Supply to sanitary appliances				√		√
Landscape irrigation of urban areas (parks, sports grounds and similar)	√	√	√	√	√	√
Street cleaning			√	√		√
Soil compaction			√			
Fire hydrants			√	√ *		√
Industrial washing of vehicles				√		√
Irrigation of crops eaten raw	√	√	√	√	√	√
Irrigation of crops not eaten raw	√	√	√	√	√	√
Irrigation of pastures for milk or meat producing animals		√	√	√	√	√
Aquaculture						√
Irrigation of trees without contact of reclaimed water with fruit for human consumption	√	√	√	√	√	√
Irrigation of ornamental flowers without contact of reclaimed water with the product		√	√	√		√
Irrigation of industrial non-food crops, fodder, cereals	√	√	√	√	√	√
Water process, and cleaning in industry other than the food industry			√	√**		√
Water process and cleaning in the food industry			√	√**		√
Cooling towers and evaporative condensers			√	√		
Golf course irrigation	√	√	√	√	√	√
Ornamental ponds without public access			√			
Aquifer recharge by localised percolation	√		√			√
Aquifer recharge by direct injection	√		√			√
Irrigation of woodland and green areas not accessible to the public		√	√	√	√	√
Silviculture						√
Environmental uses (maintenance of wetlands, minimum stream flows and similar)						√

* only for industrial uses.

** reclaimed water cannot be used in direct contact with food, pharmaceuticals, or cosmetic products.

In all cases, the limits for organics and microbiology are the focus, with treatment of the former often a prerequisite for the effective treatment of the latter. Physical, chemical and microbiological water quality guidelines and criteria from different resources and different end-uses of recycled water are collected in Table 4.

Table 4: Physical, chemical and microbiological water quality guidelines and criteria for different end-uses of recycled water (for the details of the values see the references).

		Water quality parameter:								**Pathogen criteria:**				**Disinfection parameter:**		
		pH	TSS	BOD	COD	TN	TP	EC	Turbidity	Tot. coliform	Faecal coliform	E. coli	Poliovirus/ surrogate	UV disinfection	Cl residual	Chlorine CT
	Type of reuse	/	mg/L					dS/m	NTU	cfu/100mL			log	mJ/cm^2	mg/L	mg/L-min
WHO (WHO, 2006)	Irrigation of ornamentals, fruit, trees, and fodder crops		≤140	≤240							≤1000					
	Irrigation of vegetables likely to be eaten uncooked		≤20	≤20							≤200					
	Toilet flushing		≤10	≤10	≤30						≤10					
Israel (Gross *et al.*, 2007)	Urban reuse		<10													

US (USEPA, 2012)	Unrestricted	6-9		≤10					≤2		0				1	
	Restricted	6-9	≤30	≤30							≤200				1	
	Environmental		≤30	≤30						2.2	≤200		5 log inact.		1	
California (CDPH, 2010)	Unrestricted								≤2	23				100		450
Western Australia (GWA, 2010).	Subsurface irrigation		<30	<20												
	Surface irrigation		<30	<20								<10				
	Toilet flushing		<10	<10								<1				

Canada (Health Canada, 2010)	Toilet flushing		<20										5 log			
EU (2018/0169 (COD))	Agricultural irrigation		≤10, 91/271/EEC**	≤10, 91/271/EEC**					≤5			≤10-1000**				
Cyprus (K.D.P.269/2005, KDP 772/2003)	Irrigation of vegetables likely to be eaten cooked		≤15	≤15							≤100					
	Irrigation of vegetables likely to be eaten uncooked		<10	<10							≤15					
	Range from most to least restricted reuses	6.5-8.5	10-30	10-70	70	15	2-10	1.7-2.9				$5\text{-}10^{3}$				300
France (JORF num.0153, 4 July 2014)	Range from most to least restricted reuses		15		60							$250\text{-}10^{5}$				

Greece (CMD No 145116)	Range from most to least restricted reuses	6.5-8.5	2-35	10-25		30	1-2	3.0	2-no limit		2	5-200				350
Italy (DM 185/2003)	Range from most to least restricted reuses	6.0-9.5	10	20	100	15	2	3.0				10				250
Portugal (NP 4434 2005)	Range from most to least restricted reuses	6.5-8.4	60					1.0			$100\text{-}10^4$					70
Spain (RD 1620/2007)	Range from most to least restricted reuses		5-35			10*	2*	3.0	1-15			$0\text{-}10^4$				

only for aquifer recharge and recreational uses

** depending on the wastewater treatment level, crop type and irrigation method

4 GREYWATER CHARACTERISATION

4.1 What is greywater?

Various descriptions of greywater exist; however, in general this water is characterized as a lightly polluted household wastewater in industrialised countries discharged from dishwashers, showers, sinks, bath and washing machines excluding wastewater from toilets (Department of Health Western Australia, 2010; Environment Agency, 2011; Eriksson *et al.,* 2002; World Health Organization 2006; Friedler and Hadari 2006). Some sanitary experts define greywater as water that is of less quality than potable water (drinking water), but of higher quality than blackwater (Jamrah *et al.* 2011). Wastewater from bathroom, showers, and tubs and clothes washing machines sources is termed as light greywater (Friedler and Hadari 2006). Whereas, greywater that includes more contaminated waste from laundry facilities, dishwashers and, in some instances, kitchen sinks is called as dark greywater (Figure 2) (Birks and Hills 2007). Greywater is called so because it turns grey when stored for a while without treatment.

4.2 Amount of greywater in wastewater

Household wastewater is mainly divided in blackwater and greywater. Blackwater consists of the discharges from toilets. Especially when collected with vacuum toilets, blackwater contains nitrogen and phosphorous in high concentrations and most of the pathogens, hormones and pharmaceutical residues (Zeeman *et al.* 2008). In residential buildings blackwater generation is generally a low fraction, less than 30% of the total, and even less in non-residential buildings (commercial, offices, etc.) (Li *et al.* 2009; Scheumann *et al.* 2009). Whereas greywater, normally accounts for about 65–75% of the wastewater volume produced by households, and over 90% if vacuum toilets are installed (Jamrah *et al.* 2011, Leal *et al.* 2011, Hansen and Kjellerup, 1994). Further light greywater is around 50% of the total greywater. (Leal *et al.* 2011). However, the amount and composition of greywater produced strongly depends on domestic use of fresh water of an average person which is different around the globe (see section 2.1). Data about greywater quantity produced by different sources in comparison with total wastewater produced in different cities around the world have been collected by Ghaitidak and Yadav (2013) and Friedler, (2004), which can be seen in the Table 5. Friedler, (2004) analysed different sources of greywater (Table 5) and reported that of all appliances kitchen sink was signalled out as the major greywater producer (with 13–25 L/PE/d), while the dishwasher as the least contributor (with only 2–6 L/PE/d). However, concentrations of pollutants may differ significantly among the discharges from different appliances (see chapter 4.4 for explanation and Table 6).

Table 5: Distribution of greywater at different sources in L/PE/d (Ghaitidak and Yadav, 2013 and the references therein; Friedler, 2004 and the references therein: Oron, 2014)

	Washbasin	Shower	Bath	Washing machine	Kitchen sink	Dishwasher	Toilet	Total GW	Total WW	% of GW in WW
North America [b]	90		-	82	27		-	196	378	52%
England and Wales [b]	33		-	-	63		-	96	150	64%
Jordan (Aman) [b]	20		-	10	29		-	59	75	79%
Oman [b]	83		-	13	64		-	161	195	83%
Queensland [b]	60		-	35	-		-	95	-	-
Canbera [a, b]	60		-	40	17		-	117	173	67%
New south wales [a, b]	73		-	45	15		-	133	195	23%
Yemen (Sana'a) [a, b]	17		-	5	13		-	35	40	87%
Israel [b, c, d]	15	20	34-55	8-13	25	5	50-60	98	-	-
Literature data [c]	8-13	12-20	16	17-60	13-19	2-6	-	68-134	-	-
% in GW [b]	47%			26%	27%		-	-	-	-

Qww wastewater flow, Qg quantity of greywater flow
[a] Converted to L/PE/d assuming three persons in a family
[b] (Ghaitidak and Yadav, 2013)
[c] (Friedler, 2004)
[d] (Oron *et al.,* 2014)

4.3 Sources and contents of greywater

Greywater comes from different sources, mainly from bathroom 47%, kitchen 27%, laundry 26% (Figure 1) and contains different contents (Figure 2).

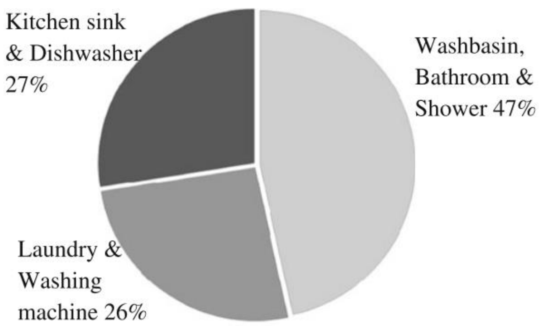


Figure 1: Distribution of greywater at different sources. (Ghaitidak and Yadav, 2013 and the references therein)

Bathroom greywater contains soaps, shampoos, toothpaste, body care products, shaving waste, skin, hair, body fats, lint, and traces of urine and faeces (Noah 2002). Greywater originating from kitchen sink contains food residues, high amounts of oil and fat, dishwashing detergents (Morel and Diener 2006; Queensland Government 2008). Greywater originating from dishwasher contains bacteria, salinity, soaps, hot water, food particles, odour, foam, high pH, oil and grease, organic matter, suspended solids, and turbidity (Noah 2002). Laundry greywater contains high concentrations of chemicals from soap powders (such as sodium, phosphorous, surfactants and nitrogen), paints, bleaches, solvents, oils, and non-biodegradable fibres from clothing (Morel and Diener 2006). Greywater from automatic clothes washer contains high pH, foam, hot water, bleach, nitrate, oil and grease, suspended solids, salinity, soaps, phosphate, sodium, and turbidity (Noah 2002). The content of these sources has already been analysed by different researchers and collected in the Figure 2.

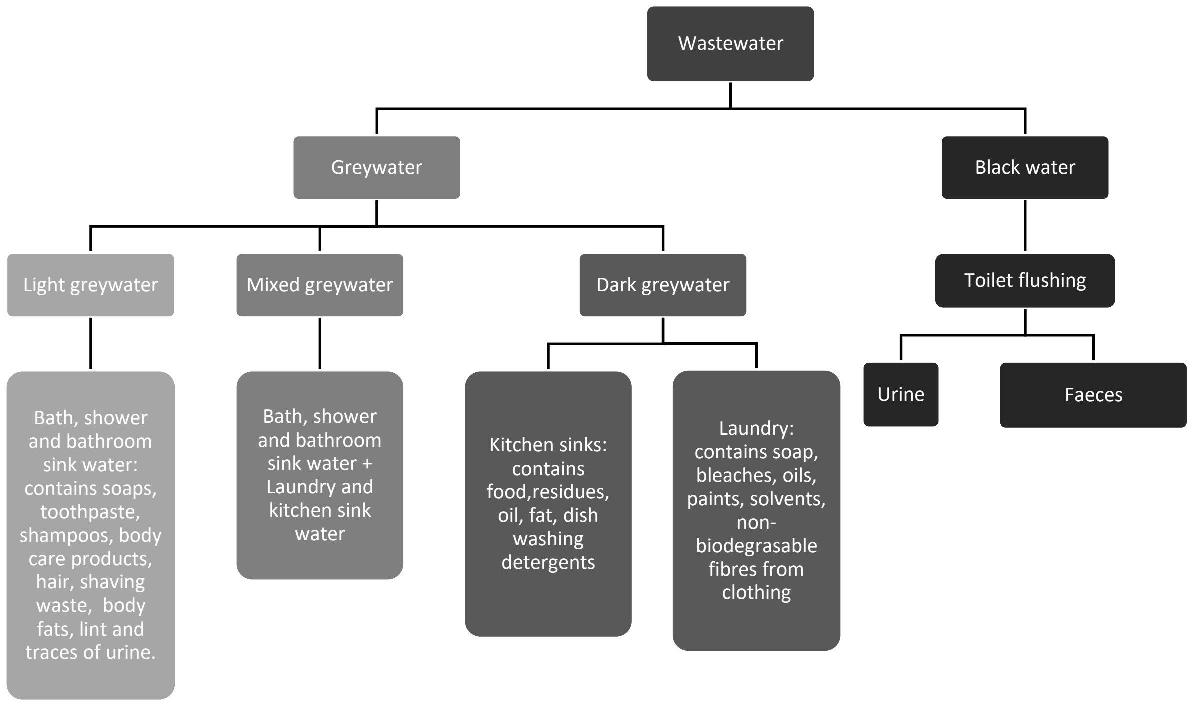


Figure 2: Greywater sources and their contents. (References: Abu Ghunmi *et al.,* 2011; Eriksson *et al.,* 2002; Ghaitidak and Yadav, 2013; Li *et al.,* 2009; Mayer *et al.,* 1999; Morel and Diener, 2006; Noah, 2002; Queensland Government, 2008)

4.4 Composition of greywater

Greywater is increasingly being considered as a viable alternative water source for non-potable water reuse because it is consistently generated in large volumes close to demand. There are inherent properties of greywater that favour its reuse in many applications. For example, greywater contains far less nitrogen compared with blackwater, leading to less vigorous treatment prior to reuse. Yet, it contains nutrients at concentrations that present a health threat to aquatic ecosystems, thus needed to be treated (Eriksson *et al.,* 2002; Boyjoo *et al.,* 2013). Proper characterization of wastewater is essential to defining the treatment to be applied. The composition of greywater varies greatly upon factors such as the quality of the source water, the number of occupants, the age distribution of the occupants, activities, their lifestyle, water usage patterns, living standards, social and cultural habits, type and quantity of household chemicals (i.e. soaps, toothpastes, shampoos, detergents, etc.) used, and length of time for which greywater is stored before being used (Hawaii State Department of Health 2009; Eriksson, 2002). In addition, concentrations of pollutants or greywater composition, may differ significantly among the discharges from different household appliances. To illustrate, Ghaitidak and Yadav (2013) collected data of characteristics at different sources seen in Table 6. This table reveals that dishwasher as the least amount of greywater contributor (with only 2-6 L/PE/d), can produce more BOD (up to 4450 BOD mg/L) concentrated greywater than kitchen sink (up to 890 BOD mg/L) which signalled out as the major greywater producer (with 13–25 L/PE/d) according to Friedler (2004). Furthermore, there are differences in both COD and BOD between various sources of greywater. Bathroom greywater levels are usually reported to lie in the range 184–633 mg/L COD and 76–300 mg/L BOD; kitchen 26–1380 mg/L COD and 5–1460 mg/L BOD; and laundry greywater 725–1815 mg/L COD and 48–472 mg/L

BOD (Nolde, 1999; Eriksson *et al.,* 2002). Friedler (2002) reported greywater concentrations of the washing machine, dishwasher and kitchen sink with COD concentration in the order of 1300 mg/L, BOD up to 700 mg/L, phosphate up to 500 mg/L, and chlorides and sodium in the range of 600–700 mg/L. However, all these values slightly differ from the ones collected in Table 6 by Ghaitidak and Yadav (2013).

Table 6: Greywater composition at the outflow of different appliances (References collected by Ghaitidak and Yadav, 2013: Fridler (2004); Jefferson *et al.* (2004); Jamrah *et al.* (2008); Jamrah *et al.* (2011); Kotut *et al.* (2011); Prathapar *et al.* (2005))

Parameters	**Wash basin**	**Bathroom**	**Shower**	**Laundry/washing machine**	**Kitchen sink**	**Dish washer**
pH	7–7.3	7.1–7.6	7.3–7.5	8.3–9.3	6.5–7.7	8.2–8.3
Turbidity (NTU)[a]	164	59.8	84.8–375	328–444	133–211	
EC (mS/m)[a]		43.7	1.4–89	2.9–703	1.4–97	90.61
Tot. solids (TS)	835	777	520–1,090	2,021–2,700	679–1,272	2,819
Tot. suspended solids (TSS)	153–259	58–78	89–353	188–315	134–625	525
Tot. dissolved solids (TDS)			279–565	2,140–2,444	312–903	
BOD	155–205	129–173	40.2–424	44.3–462	40.8–890	470–4,450
COD	386–587	230–367	77–645	58–1,339	58–1,340	1,296
Tot. Alkalinity			203	333.6	205.4	
Chlorides	237	166	147–284	205–450	158–223	716
MBAS	3.3	15	14.9–61	42–118.3	41.9–59	11.1
O&G	135	77	164	181	232	328
Tot. N	10.4	6.6	8.7–10.92	14.28	6.44–6.44	
Tot. P			1.12	51.58	0.69	
TC (MPN)[a]	9.42E3	6,350–5.1E6	2E2–6.8E3	2E2–4.2E6	2E2–5.29E2	4.30E6
FC (MPN)[a]	3.50E4	1.5E5–4E6	64–4.0E6	13–4.E6	200.5–1.2E6	6.0E4–3.2E5
E. coli (MPN)[a]	10	82.7	2E2–1.49E3		2E2	
Boron (B)	0.44	0.41	0.35–0.35	0.4	0.02–0.02	3.8
Calcium (Ca)			15.7–59.9	18.7–24	19.69–23.6	
Magnesium (Mg)			23–56.1	15.1–60.8	16.6–21	
Sodium (Na)	131	112	109.5–184.5	302.1–667	70.1–148.9	641
Arsenic (As)			0.03		0.015	
Copper (Cu)			0.01–0.0127	0.0064–0.01		
Lead (Pb)			0.1036	0.0829	0.0622	
Nickel (Ni)			0.035	0.0352–0.12	0.0352–0.04	
Zinc (Zn)			2.4	0.14	0.039–0.04	

[a] Units in bracket; all other units are in mg/L (except pH)

Therefore, it should be noted that characteristics vary from one research paper to another, thus accentuating the different range of greywater concentrations. These findings illustrate that the different types of greywater could be suitable for different types of reuse, and there will be different needs for pre-treatment depending on both the types of greywater and the intended use of the water (Eriksson *et al.*, 2002). Greywater can be lightly to heavily polluted and the values of specific parameters can reach almost to an extent similar to mixed wastewater. In Table 7 those parameter ranges of physical, chemical and microbiological water quality of typical greywater, wastewater streams, drinking water and disinfection are presented and compared.

Table 7: Physical, chemical, and microbiological water quality of typical greywater, wastewater streams, drinking water and disinfection influent ranges.

Parameter	Units	Drinking water	Light GW		Mixed GW					Mixed WW	Disinfection
Reference		a	b	c	b	d	e	f	g	b	b
Physical and chemical parameters											
BOD	mg/L		20–240		5–1056	41.2	90-290	5-890		112–1101	1.1–62
COD			100–633		58–2950	78				1329–1650	17–130
Ammonia							<0.1-25.4				
Ammonium		0.5						0.002-25			
Nitrite							<0.1-0.8				
Nitrate								0-6.3			
Total N		0.5	3.6–19.4		1.1–74		2.1-31.5			9.0–240	2.8–4.1
Phosphate								1-170			
Total P			0.11–48.8		0.062–500		0.6-27.3			0.2–32	
TSS			29–505		19–700	168	45-330			22–1690	4.0–32
DO		5				8.5					
pH						7.81			5-8.7		
Turbidity	NTU		12.6–240		19–444	48.9	22-220				0.2–35
EC	µS/cm	2500		300-1500			325-1140				
Bacteria and bacterial indicators											
Total Coliform	CFU/100 mL	0	1–7.4		3.1–8.8					7.0–9.0	2.0–5.8
Fecal Coliform			0–6.9		2.0–8.0					4.0–8.2	1.4–5.1
E. coli			2.3–5.7		3.6–6.7					4.0–7.9	2.6
Enterococci		0	1.9–3.4		2.4–4.6					4.0–5.0	1.8–3.8
P. aeruginosa			2.6–3.5		2.3–4.3					3.0–6.0	2.1–3.8
S. aureus			4.0–5.7		3.3–5.7						1.4–1.9
C. perfringens		0	0.66							3.0–5.0	/
Legionella			2.2		2.2–2.9						/
Salmonella					3.7					2.0–4.0	/
Viral indicators											
MS2-Coliphage	PFU/mL				3.0–4.0						5.6–8.2

a (Directive 98/83/CE_EN)
b (Arden and Ma, 2018)
c (DWAF, 1996; Morel and Diener, 2006)
d (Jamrah, A. *et al.* 2011)
e (Jeppesen and Solley, 1994)
f (Castellar Da Cunha, J. A. *et al.* 2018)
g (Christova-Boal *et al.,* 1996)

4.4.1 BOD and COD in greywater

Concentrations of both biochemical oxygen demand (BOD) and chemical oxygen demand (COD) in greywater are derived from household chemicals such as dishwashing and laundry detergents, food waste from the kitchen sinks, and body dirt in the bathtub and laundry (Weston, 1998). Concentrations of COD and BOD differ between light and mixed greywater (Table 7). Mostly light greywater is produced in the hotels and mixed greywater is produced in residential homes. The high COD concentrations of 724 mg/L in mixed greywater were reported by Otterpohl (2002). The values contradicted the general belief that greywater is very diluted compared to sewage. In fact, it contains approximately 50% of the COD discharged by households (Otterpohl, 2002). Another sampling campaign by Friedler (2002) also revealed that in contrast with the common perception, domestic greywater was found to be quite highly polluted. The specific daily BOD loads of greywater were found to be 47 g/PE/d. These comprise 55-70% of the "common" specific load of BOD in municipal sewage. The larger values for BOD and COD are often attributed to heavy detergent or food waste loads associated with laundry or kitchen sources (Ghaitidak and Yadav, 2013), and can be particularly extreme if unmixed with more dilute sources, even exhibiting similar or greater concentrations than mixed wastewater. Moreover, compared to the oxygen demand, mixed wastewater is more readily degradable due to faecal material and food waste, that of less biodegradable greywater due to surfactants from soaps and detergents (Christova-Boal *et al.,* 1996; Sharvelle *et al.,* 2007). Table 8 shows the characteristics of greywater from several selected studies collected by Hernández *et al.* (2011). This table excludes those studies on low-strength greywaters, which exclude laundry and kitchen sink discharges.

Table 8: Characteristics of greywater in different locations (Hernández *et al.,* 2011 and the references therein)

Greywater source	Sampling	COD	BOD	Total N	NH_4^+–N	Ortho P	Total P	AS
111 houses, D	4 months	258–354		9.7–16.6			5.2–9.6	
111 houses, D	9 months, n = 6	640		27.2	4.2	8	9.8	
37 houses, S	2 months, n = 8	361		18.1			3.9	
47 houses, S	n = 4	588		9.7			7.5	
150 houses, NL	2 weeks, n = 104	425	215	17.2	7.2	2.3	5,7	
32 houses, NL	4 months, n = 10	1583		47.8	16.4	2.3	9.9	
81-room-hotel, E	1 year, n = 24	171		11.4				
6 person-farm, IS	9 months, n = 72	686	270	14			18	40
House 1, IS	1 year, n = 96	474	195					17
House 2, IS	1 year, n = 96	200	62					3
6 houses, IS	5 weeks, n = 5		133	19			31	34
13 families, J	n = 6	1351	873	17				76
Villages, SA	n = 100	4770		72				
University, SA	Not indicated			206	157	40	69	
4 houses, CR	1 year, n = 11		167			6.28		
One family, USA	n = 10			0.6–5.2	0.12–2.49	1.9–16.9		

*all values are presented in mg/L, AS = anionic surfactants, D = Germany, S = Sweden, NL = The Netherlands, SA = South Africa, E = Spain, J = Jordan, IS = Israel, CR = Costa Rica.

Hernández *et al.* (2011) reported values for COD from 171 to 4770 mg/L. Very diluted greywater is usually obtained from hotels (e.g. 171 mg/L of COD from a hotel in Spain), probably due to the higher water consumption in hotels (in Europe it is estimated between 170 and 360 L per guest-night)

(Bohdanowicz, 2005). High COD values of 1352 and 4770 mg/L can be related to low water consumption (due to scarcity) such as in the rural areas of Jordan and South Africa. Although, greywater in some cases may be highly polluted, with faecal coliforms of about 104 -108 CFU/100 mL, COD concentrations of up to a 1,000 mg/L, and significant concentrations of detergents and salts (boron, sodium and chlorides) etc. (Almeida *et al.,* 1999; Diaper *et al.,* 2001; Dixon *et al.,* 1999; Patterson *et al.,* 2001; Rose *et al.,* 1991).

4.4.2 Oxygen in greywater

The quantities of dissolved oxygen in greywater have been measured by Shin *et al.* (1998) and Santala *et al.* (1998) who found concentrations in the ranges 2.2–5.8 and 0.4–4.6 mg/L, respectively.

4.4.3 Nitrogen in greywater

The total nitrogen concentration of the greywater is lower than in domestic wastewater, 0.6–74 and 20–80 mg/L, respectively (Eriksson *et al.*, 2002). The main source for nitrogen in domestic wastewater, urine, should not be present in greywater. The kitchen wastewater contributes the highest levels of nitrogen to the greywater (concentration range 40–74 mg/L). The corresponding values for lowest levels of ammonium are <0.05–25 mg/L in the bathroom compared to the highest values 12–50 mg/L in the laundry (Eriksson *et al.*, 2002). A study by Hernández (2011) reported greywater concentrations of total nitrogen to be about 26 mg/L, of which only 16% was inorganic, with 10% ammonium and 3% nitrate and nitrite, respectively. That means that the major part of the nitrogen was organically bound, which is contrary to the case of sewage, in which a large fraction (50–93%) of the nitrogen is present as ammonium (Elmitwalli *et al.,* 2000). This is probably due to the absence of urea from urine, which transforms very quickly into ammonium and accounts for up to 90% of the nitrogen in sewage (Hernández, 2011).

4.4.4 Phosphorus in greywater

In terms of nutrients, ranges are again large. Phosphorus concentrations can be high, particularly in areas that have not adopted stringent legislation banning the use of phosphate-based detergents (Turner *et al.,* 2013; Jeppesen, 1996). However, since the mid-eighties no phosphate is allowed in laundry detergents, therefore, the sources of phosphate in greywater may come from food processing and dishwashing liquids (Hernández, 2011). Concentrations between 6 and 23 mg Tot-P/L can be found in traditional wastewaters in areas where phosphorus detergents are used. However, in regions were non-phosphorus detergents are used the concentrations range between 4 and 14 mg/L (Henze *et al.*, 2001). This can explain why the total phosphorus and phosphate concentrations are generally higher in laundry greywater compared to bathroom greywater, 0.1–57 and 0.1–2 mg/L, respectively (Eriksson *et al.*, 2002). A study by Hernández (2011) reported greywater concentrations of total phosphorus to be similar as in other studies, where total P ranged from 3.9 to 9.9 mg/L. Another study by Kroes (1980) reported greywater concentrations of phosphorus to be 7.2 mg/L, of which 35% was in the form of phosphate and 65% was particulate phosphorus.

4.4.5 Bacteria in greywater

Similar to the physical and chemical parameters, mixed greywater faecal indicator concentrations often reach those of mixed wastewater, with ranges of up to 8 log (CFU/100 mL). Some of the main bacterial indicators present in the greywater are listed in the Table 7. Friedler (2004) reported the bath and shower to be signalled as the major sources of faecal coliforms with average concentrations of $4*10^6$ CFU/100 mL. In addition to the large variability in bacterial pathogen data, there is a lack of data regarding specific virus or protozoa counts in greywater. Similar to bacteria, adsorption onto plant and media surfaces is a primary mechanism of virus removal in wetlands (Garcia *et al.,* 2010; Jackson and Jackson, 2008), often occurring within the first hours of entering the wetland (Hodgson *et al.,* 2003). However, sedimentation is unlikely to be a significant virus reduction mechanism (Symonds *et al.,* 2014) and wetlands that rely on pathogen removal via sedimentation have shown poor virus removal performance (Falabi *et al.,* 2002). Accordingly, if used as a standalone unit process, greywater wetlands cannot reliably meet microbiological effluent standards. Results from the reviewed greywater disinfection experiments suggest that if organics are sufficiently removed from greywater, a chlorine dosage of 100 mg/L-min or UV dosage of 100 mJ/cm^2 is likely appropriate for meeting all USEPA guidelines and all Western Australia guidelines (Friedler, 2004).

4.4.6 Temperature in greywater

Eriksson *et al.* (2002) found that temperature of greywater varies within the range 18–38 °C. The rather high temperature is due to the use of warm water for personal hygiene. This relatively high temperature may cause problems since it favours microbiological growth that causes clogging. The elevated temperatures may also result in $CaCO_3$ precipitation since its solubility and some other inorganic salts decrease at elevated temperatures. Microbially-mediated processes responsible for nitrogen removal in these plant-soil systems are largely dependent on temperature, with an increase in performance typically observed with temperature (Akratos and Tsihrintzis, 2007). Temperature can also affect vegetation function since plants are sensitive to hot water (Picard *et al.,* 2005). Temperature seems to have minimal effect on BOD removal, and phosphorus sorption reactions (Kadlec and Reddy, 2001). In so far as microbial metabolic processes in wetlands often proceed at greater rates in warmer temperatures (Reddy and DeLaune, 2008), wetlands located in colder climates may have a lesser capacity to reduce BOD concentrations, and treatment performance may be diminished in winter months (see more in section 5).

4.4.7 Turbidity in greywater

Turbidity is the measure of relative clarity of a liquid. Material that causes water to be turbid include clay, silt, very tiny inorganic and organic matter, algae, dissolved coloured organic compounds, and plankton and other microscopic organisms. These particles provide attachment places for other pollutants, notably metals and bacteria. For this reason, turbidity readings can be used as an indicator of potential pollution in a water body. (Swanson and Baldwin, 1965). Turbidity of greywater is usually in the range 15.3–375 NTU (Eriksson *et al.*, 2002, Li et al, 2009). Highest turbidity can be found in laundry greywater 50–440 NTU (Li et al, 2009).

4.4.8 Redox potential in greywater

Oxidation reduction potential is the energy potential for chemical processes to neutralize contamination. It shows the minimal voltage in the water, which comes from the electric charge of reducing agents or oxidizing agents. Redox potentials of less than −100 mV indicate anaerobic environments, while values greater than 100 mV indicate aerobic environments (Hussein *et al.*, 2017). Strictly aerobic microorganisms are generally active at positive redox values, whereas strict anaerobes are generally active at negative redox values. There are organisms that can adjust their metabolism to their environment, such as facultative anaerobes. Facultative anaerobes can be active at positive redox values, and at negative redox values in the presence of oxygen-bearing inorganic compounds, such as nitrates and sulphates (Chuan *et al.*, 1996). Greywater as polluted water has negative values of redox potential and can be increased with oxygen increase or pH decrease.

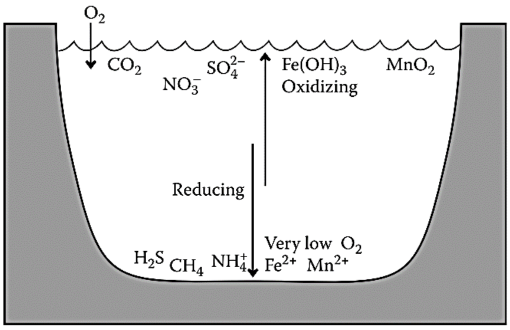


Figure 3: Predominance of various chemical species in a stratified body of water that has a high oxygen concentration (oxidizing) near the surface and a low oxygen concentration (reducing) near the bottom (Manahan, 2010).

4.4.9 pH in greywater

The highest pH is usually found in laundry water. Generally, it ranges from 8–10 pH, while other sources of greywater have lower pH values, ranging from 5–8.7. High pH of laundry greywater is the result of high concentrations of soaps, powdered detergents and softeners. It has been reported that pH affects plant and microbial growth and influences soil properties. The optimal pH range for irrigation water is 6.5–8.4, to avoid negative impacts for both soil and plants. Most plants grow best in soils with pH ranging between 5 and 7. Plant nutrients are mostly available in the pH range 5.5–6.5, which is also a good range for beneficial soil bacteria (CSBE, 2003). A change in soil pH influences several soil properties, which directly affects plant growth, soil bacteria and the availability of nutrients (Christova-Boal *et al.,* 1996; Eriksson *et al.*, 2002). When the soil pH is 5 or lower, nitrates, phosphates, and potassium become less available to plants and soil bacteria become less active (Crook *et al.,* 1994). When the soil pH is 8 or higher, iron and zinc become less available to plants, which result in development of chlorotic leaves (Rodda *et al.* 2010). In substrates that involve limestone pH of treated wastewater may increase due to its buffer capacity (Ghaitidak and Yadav, 2013).

4.4.10 Electrical conductivity in greywater

Electrical conductivity (EC) measures the concentration of dissolved salts, both positively and negatively charged ions. The most common sources of salts in greywater are sodium-based soaps, found in detergents and powdered soaps. Sodium ions, along with potassium, calcium, magnesium, and chloride ions, are the major ions contributing to soil salinity. If soil is irrigated continuously with water of high EC and if insufficient leaching occurs, salts will accumulate through the soil profile and the soil will become saline. High EC in irrigation water induce salt accumulation in soils, which reduces water uptake by plants by lowering osmotic potential and reducing plant productivity. The concentration of electrical conductivity in greywater has been found to be in the range 30 to 150 mS/m and sometimes can be as high as 270 mS/m (DWAF, 1996; Morel and Diener, 2006). A study by Ghaitidak and Yadav (2013) reported an increase of EC from 95 to 110 mS/m, which might be due to loss of water from the system through evapotranspiration, resulting in an increase in the dissolved mineral content of the treated greywater.

5 HEAT RECOVERY FROM GREYWATER

In this chapter potential heat recovery from drain water (domestic wastewater) and greywater is briefly discussed and analysed. Next, an overview of commercially small-scale heat exchangers is presented. Small scale heat recovery includes heat recovery from greywater of individual appliances such as dishwasher, washing machine and shower or, it can be designed as a centralised heat recovery from greywater collected from all household appliances combined. The latter was recognised as the most efficient and reasonable system to install in the larger (residential) buildings in the future and was targeted as a design solution for this master thesis and further work in the field of energy recovery.

Nowadays, global policies tend to move towards a more sustainable approach with a more responsible use of energy. In 2014, the European Union set the goal of reducing greenhouse gas emissions and improving the energy efficiency up to 40% and 27% respectively by 2030 (García-Álvarez *et al.*, 2016). In past years, a great effort has been made in order to reduce the energy consumption in buildings. Passive houses linked to reduction of heating demand and the improvement of technologies used for heating and cooling have been some of the focal points. In addition, the Energy performance of buildings directive 2010/31/EU (EPBD) requires all new buildings from 2021 (public buildings from 2019) to be nearly zero-energy buildings (NZEB). "Nearly zero-energy building" is a building that has a very high energy performance, requires the nearly zero or very low amount of energy and is covered by energy from renewable sources, including energy from renewable sources produced on-site or nearby (Magrini *et al.* 2020). EU programs, specifically "Horizon 2020", promote the NZEB design and also its evolution, namely the Positive Energy Building (PEB) model. EU Commission Recommendation, (2019) notes the importance of energy recovery in building sector: "Buildings are central to the Union's energy efficiency policy as they account for nearly 40% of final energy consumption", of which 27% is attributable to the residential sector. The residential sector accounts for 29.66 MWh/yr, 26% of the total consumption in USA respectively. Of this, 37% energy is electricity for lighting, cooling, and appliances. The remaining energy used is attributed to heating: 45% for space heating and 18% for water heating (DOE, 2011). In other countries like the UK, the residential sector represents 30% of total energy consumption in the country. Of this, 65% energy is used for space heating and 20% for water heating only (Druckman and Jackson, 2008). Therefore, domestic hot water (DHW) accounts for 4–6% of the total national energy demand.

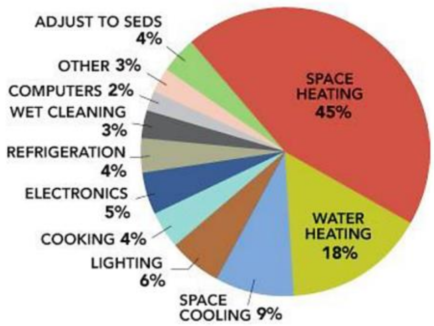


Figure 4: U.S. residential site energy demand by end-use in 2011 (U.S. Department of Energy, 2012)

The Swedish Energy Agency (2009) has estimated the heat requirement in households in Sweden for DHW of 780 to 1150 kWh/cap/yr. These statistics are often used to argue DHW savings and to motivate recovery of heat from wastewater. Therefore, the reduction of the energy consumption and the improvement of the technologies used in this sector are necessary in order to reach the 2030 EU targets. To close the energy loop of the building sector, heat recovery from hot drain water (wastewater) that was officially recognized as a renewable energy source by the European Parliament (European

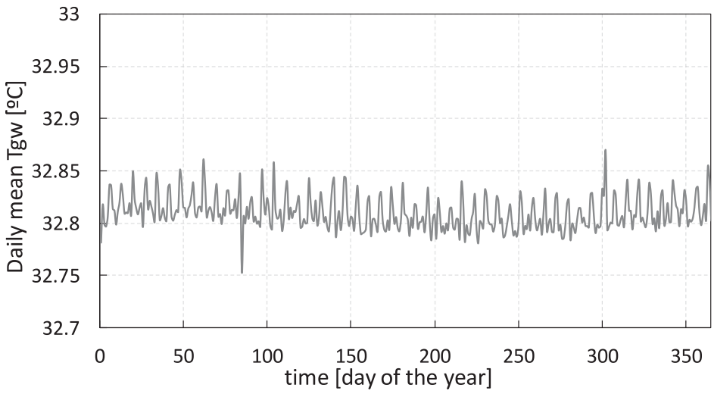
Parliament and Council of the European Union, 2018), is some of the energy recovery systems that are taking its reputation nowadays due to the major part of the electric bill occupied by heating domestic water (Ramadan *et al.*, 2016; Hervás-Blasco *et al.*, 2020).

Reasons to recover heat from domestic wastewater

Before searching for new sources of energy, already available waste and excess energy sources should be considered as a prior alternative source of energy such as wastewater. There are many reasons to recover heat from domestic wastewater. For example, in America, typically 80–90% of the energy used to heat water in a home goes back down the drain without recovery, this means that year after year several kWh of heat is flushed down the sewer system and in this regard the costs for DHW and space heating are not getting any lower (Henderson and Hewitt, 2001). Moreover, in the USA, many municipalities have an upper limit of temperature 49–60 °C, on drain water entering the sewer system (Arnell *et al.*, 2017). This conveys that many larger facilities such as hospitals that have laundry or kitchens, might have to cool their drain water. In this concern, heat recovery would seem appropriate solution.

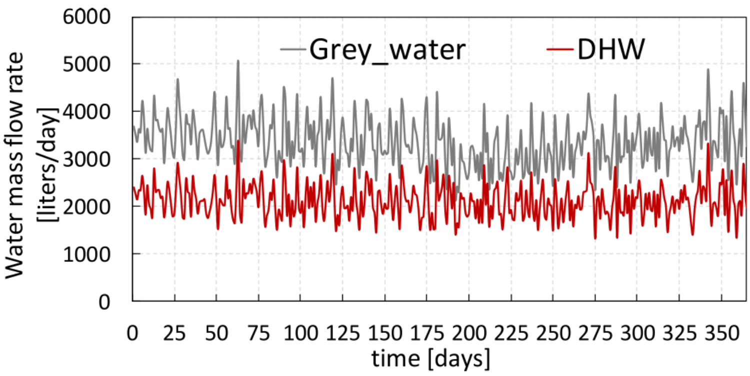
5.1 Temperature of wastewater

Household drain water has a more suitable temperature for heat recovery than other heat sources like sea, lake water or groundwater (Seybold and Brunk, 2013). In fact, the usual temperature of drain water in households and residential buildings ranges from 20 °C to 40 °C (McNabola and Shields, 2013) or from 23 °C to 26 °C (Seybold and Brunk, 2013). Daily mean temperature of greywater from 20 dwellings was collected by Hervás-Blasco *et al.* (2020) for a whole year, with seasonal variations.



Graph 2: Daily mean greywater temperature over a year (Hervás-Blasco *et al.*, 2020)

About 60% of the total water quantity is hot water. However, hot water use can be very different and ranges from 5.5 to 25.1 m3 per person. The amount of greywater produced in 20 dwerlings in comparison to DHW usage over a year was presented in the study from Hervás-Blasco *et al.* (2020). Hot water apeared to account roughly two thirds of the total greywater production which includes both cold and hot water after their use.



Graph 3: DWH and greywater daily-profiles for 20 dwellings DWH consumption at 45°C and Tnet 10°C (Hervás-Blasco *et al.*, 2020)

The results from the study Johansson *et al.* (2007) showed the following division of tap water use in a residential house:

- Wash basin 16% (10% hot water and 6% cold water)
- Dishwashers 1% (1% cold water)
- Laundry 18% (18% cold water)
- Kitchen sink 33% (23% hot water and 10% cold water)
- Shower/bathtub 32% (27% hot water and 5% cold water)

Kleven (2012) collected the data of DHW use in a building with 300 m^2 living area and a total of 18 occupants and presented it in kWh.

Table 9: Hot water use in a building with 300 m^2 living area and a total of 18 occupants (Kleven, 2012)

Hot water use		**Energy use [kWh/day]**
Hot water boiler	Shower	5.17
	Sink in Bathroom	3.45
	Sink in kitchen	0.96
Washing machine		0.8
Dishwasher		1.87
Total		12.25

Table 10 by Hervás-Blasco *et al.* (2020) shows the temperatures at the end-use and at the drain considered in this study. Regarding the temperature of the greywater at the drain, a drop of 7 K from the end-user temperature was estimated regardless the nature of the consumption (Nehm *et al.*, 2008). An average greywater temperature of 32.8 °C were calculated out of that mix.

Table 10: End-use water temperatures and drain temperatures of the different streams considered in the study by Hervás-Blasco *et al.* (2020)

Draw-off type	**End-use temperature [°C]**	**Drain temperature [°C]**
Handwashing	38	31
Shower	40	33
Bathtub	40	33
Cooking/cleaning	45	38
Washing mashine	37	30
Dishwasher	53	46

The drain water temperature from showers or washing machines is substantially higher. Letting this amount of energy leave the building with warm water without reusing it, represents a great inefficiency in the energy system. This has opened an industry for technologies aiming to reduce the energy demand for heating water, such as by heat recovery from drain water (Hervás-Blasco *et al.*, 2020).

5.2 Efficiency of drain water heat recovery systems

Energy recovery consists in recuperating the waste energy, present in many systems, and reutilizing it in a useful way. The drain water heat recovery system (DWHRS) is designed to recover the residual energy from the warm or hot drain water, and then use it to preheat the incoming cold water. This technology is an efficient and cost-effective way of recovering heat for its reutilisation as space and sanitary hot water heating (Torras *et al.,* 2016).

Heat can be recovered from domestic wastewater at the small scale in residential houses, at the medium scale in sewer networks and at the large scale in wastewater treatment plants (Frijns *et al.*, 2013). In small-scale wastewater systems, waste heat can be recovered using a DWHRS and used to preheat the incoming cold water to the electric water heater during a shower as waste heat is generated and hot water is required (Bertrand *et al.*, 2017). On the other hand, waste heat can be recovered by collecting greywater in a centralised heat recovery system that preheats fresh sanitary water that enters a boiler for hot water or preferably the much more energy efficient heat pump.

Various studies have been carried out to analyse DWHRS. Depending on the system, 30% to 75% of the heat from drain water can be recovered (Zaloum *et al.*, 2007b). In a study by Proskiw in 1998, it was concluded that more than 50% of the DHW load can be recovered. In a study conducted on different DWHRS by Zaloum *et al.* (2007a), it was observed that up to 16% of energy in the form of gas used by DHW heating systems can be saved. Waste heat recovery from drain water in high rise building was investigated by Wong *et al.* (2010), where a horizontal counter flow heat exchanger was utilized to extract heat and use it to heat cold water. Authors showed that by installing heat recovery system up to 15% of the wastewater heat can be recovered. Heat recovery from home appliances such as dishwashers was investigated in Paepe *et al.* (2003). The dishwasher proved to be a valuable source to the greywater heat recovery unit since the temperature in dishwashers were found in the area of 55 °C and above which showed to be economically beneficial (Kleven, 2012). The installation cost of the small scale DWHRS ranges from $300 to $500 and with a payback period of 2.5 to 7 years for a 3-person household, depending on how often the system is used (Słyś and Kordana, 2014). A study by Bartkowiak and Hair (2009) found that by installing the shower heat exchanger (Hotshot), families heating the DHW with natural gas could save approximately $71 per year, while the families heating the DHW with electric water heaters could save approximately $160. A study by Ip *et al.* (2018) concluded that for a shower usage of 1050 minutes per week the payback period is about 2.5 years, whereas for a shower usage of 72 minutes per week the payback period does not fall within the lifetime of the DWHRS. Hervás-Blasco *et al.* (2020) addressed the potentiality of the wasted heat from greywater as a heat source to produce DHW based on a heat pump system. In the experiment 20 dwellings were included. The obtained results showed that with the proper sizing and control of the system, a relatively small heat pump (6–12 kW) with a variable volume DHW tank of 500 l is able to satisfy the required DHW demand of 20 dwellings and by recovering 80% of the available recovery heat, the total demand of DHW is satisfied with high levels of comfort and efficiency (Hervás-Blasco *et al.*, 2020). Next, Wallin and Claesson (2014b) studied experimentally the performance of DWHRS in a heat pump system. In their work, the heat exchanger had the capacity to recover more than 25% of the energy available in the drain water for the investigated flow rates (as an evaporator to recover waste heat from shower water).

5.3 Drain water heat recovery systems working mechanism

The idea of Drain Water Heat Recovery Systems (DWHRS) is to efficiently recover thermal energy of wastewater leaving the building to heat fresh sanitary water used in the building. The drain water transfers a part of its thermal energy to the incoming cold sanitary water in a heat exchanger. The preheated incoming water is then heated to the required temperature level in a heater. Preheated incoming sanitary water has a higher temperature than the cold incoming water from water distribution system therefore, less energy is needed for the electrical heater. Other options include using the preheated sanitary water in a boiler or a heat pump (Hamann, 2015). Numerous factors directly and indirectly associated with the heat recovery system impact the function of the system. Factors directly associated with the heat recovery system, such as system design and geometry, configuration, water characteristics, the number of inhabitants and their usage patterns affect the amount of energy recovered from the drain water (Cipolla and Maglionico, 2014; Słyś and Kordana, 2014; Bertrand *et al.*, 2017). Studies found that the incoming cold-water temperature (Henderson and Hewitt, 2001), drain water temperature, the amount of drain water (Słyś and Kordana, 2014), and the characteristics of the heat exchanger (Zaloum *et al.*, 2007a, 2007b) have a significant influence on the energy recovery from the DWHRS. For instance, the study of Henderson and Hewitt (2001) evaluated the energy savings for different volumes of drain water and found a linear relationship between the two. Henderson and Hewitt (2001) found that the percentage of energy recovered increases from 73% to 80% when the drain water temperature increases from 30 °C to 60 °C. Słyś and Kordana (2014) found that the energy recovered and the net present value of the DWHRS increases with an increase in the amount of drain water. Zaloum *et al.* (2007a) reported that the highest efficiency could be achieved with a counter flow set-up of the hot and cold water streams when the movement of the two media is simultaneous. Zaloum *et al.* (2007b) conducted an experiment on eight different copper tube heat exchangers (tube pipe coiled around the drain pipe, see Figure 5) from four different manufacturers to determine the effect of flow rate, temperature and configuration in heat recovery performance. The maximum heat recovery was reached at the lowest flow rates (using the same flow rates for the two media), while the other parameters showed no significant effect on the heat recovery efficiency. Zaloum *et al.* (2007b) evaluated the number of transfer unit (NTU) effectiveness of eight heat exchangers and found that the length of the heat exchanger, the number of passes and the squareness of the tubes affect the energy recovery from the DWHRS. Also, the study found that tightly coiled tubes, without air spaces, increased the heat transfer efficiency between hot drain water in the central piped coiled by a tube pipe filled with cold sanitary water, as air is not a good conductor of heat (Zaloum *et al.*, 2007a, 2007b). A study by Benntjes *et al.* (2014) showed that the effectiveness of heat recovery system deceases with the flow rate and that there is a critical flow rate below which the performance cannot be extrapolated. Parametric study by Wallin and Claesson *et al.* (2014a) on drain water heat recovery using inline vertical heat exchanger for several flow scenarios showed that the amount of recovered heat highly depends on the sizing of the heat recovery system.

5.4 Obstacles of heat recovery

Constructing hot water systems in buildings requires considering regulations and norms to provide safe drinking water after opening the faucet, as well as minimizing the energy loss in the heat transfer. According to Slovenian recommendations, the temperature of hot water must be 55 °C ± 5 °C (Komunalno podjetje Velenje, 2020) and according to the Swedish standards, it must be at least 50 °C at the tapping point as well as in the hot water circulation (HWC) system in order to prevent any bacteria growth especially Legionella outbreaks (Olsson, 2003), which its optimal growth temperature lies between 25 and 42°C. HWC systems are built with the purpose to deliver hot water on tap quicker upon request, thereby saving water. This demands that the hot water, coming out from the water heater, must be higher. However, the water at the tapping point must be below 65 °C to avoid scalding (Olsson, 2003). Next, colder tap water increases the cost of heating. In Sweden, the water source temperatures vary between 4–15 °C and in Slovenia it is about 10 °C however, it rises up to 20 °C in some areas in summer (NIJZ, 2014). It is important that the cold-water temperature does not exceed 20 °C, with 25 °C being the bare maximum, due to bacterial growth (WHO, 2017) which should be considered with greywater when stored. Some other obstacles for utilizing heat exchangers are for example: clogging and fouling of equipment, potentially negative system impacts and economic feasibility. (Arnell *et al.*, 2017). Grundén and Grischek (2016) found that fouling significantly increases the thermal resistance of aluminium pipes. Secondly, corrosion causes a significant decrease in the pipes' thermal resistance. The combination of these effects led to an increase of 14% in thermal resistance in the examined system after three years compared to the time of installation. The increase in thermal resistance due to corrosion in the test pipe was 44% compared to the time of installation. Furthermore, the thermal resistance of the test pipe increased by 51% when it was cleaned from the fouling. The fouling resistance of the 0.81 mm fouling layer was found to be 0.03068 m2K/W. Moreover, the loss of material due to corrosion was measured to 183 g, 4.5% of the assumed initial mass of the test pipe. These facts and figures form some boundaries also for the implementation of heat exchangers on mixed wastewater and greywater.

5.5 Types of Heat exchangers

Several types of heat exchangers can be found on the market, such as tube, plate, plate-and-shell, plate-fin, spiral heat exchanger and many more (de Vries, 2015). When it comes to recovering heat from wastewater however, the approach of heat exchange is quite underutilised and disputed. The concept of drain water heat recovery system (DWHRS) is a relatively new idea and has been around for since the 1980s (US patent number 4,304,292).

Showering constitutes about 17–32% of the total water consumption in a residential house (Mayer *et al.*, 1999; Johansson *et al.* 2007). Therefore, several small companies found an opportunity to offer compact heat recovery system and focused on the heat exchangers for recovering heat from showering drain water. Companies that specialize in developing heat exchangers for specific waste heat recovery applications in wastewater heat recovery are for example EcoDrain, HeatSnagger, HXdrain, Showersave, Meander heat recovery (VX-Pipe), GFX, ReTherm, and Renewability Energy Inc. (Power-Pipe). Most of these companies specialize in retrofitting heat exchangers but might face major design constraints during implementation. Authors like Cooperman and Dieckmann (2011) classified DWHRS into two types: on-demand and storage. For the on-demand type of DWHRS it is referred to as a gravity film heat exchanger such as field-tube heat exchanger and coiled pipe. The heat exchangers can be installed either horizontally or vertically. Vertically installed heat exchangers in showers allow for heat transfer along the full boundary of the pipe wall and show heat transfer efficiencies of about 75% (Henderson and Hewitt, 2001), while the horizontally installed heat exchangers such as flat plates provide a lower efficiency of approximately 50% due to the use of only a portion of the pipe wall for heat transfer (McNabola and Shields, 2013). Heat exchangers with coiled pipe are used in many applications such as the nuclear industry, refrigeration, and food industry. And the reason behind this is because the coiled pipe has higher heat transfer coefficients in comparison with a straight tube (Prabhanjan *et al.*, 2002), while it also allows a more compact structure. The working mechanism of a coiled pipe is very simple (see Figure 5). The outgoing hot drain water flows down through an inner pipe with larger diameter while the cold ingoing water flows up in the external smaller diameter coiled tube, wrapped tightly around the drain pipe. The drain water runs down the wall of the inner pipe as a falling film. This provides an optimal contact with the inner surface allowing heat transfer to the water in the coiled pipe. This system is used by different companies that produced products such as GFX, the

Power-Pipe, and the ReTherm. The efficiencies of each of these products were determined in a series of tests run by the Canadian Centre for Housing Technology (Zaloum *et al.*, 2007b). The results for the tests are summarized in Table 11. Copper Coil Heat exchangers require to be quite long in order to maximize efficiency (models range from 30 inches to 60 inches). Additionally, each of these products reroutes water to a storage tank and are not compatible with tank-less water heaters.

Table 11: Competitive 60'' copper coil DWHR efficiency and price (Zaloum *et al.*, 2007b)

Model	Efficiency
Power Pipe R3-60	55%
GFX	48%
Retherm SC-60	43%

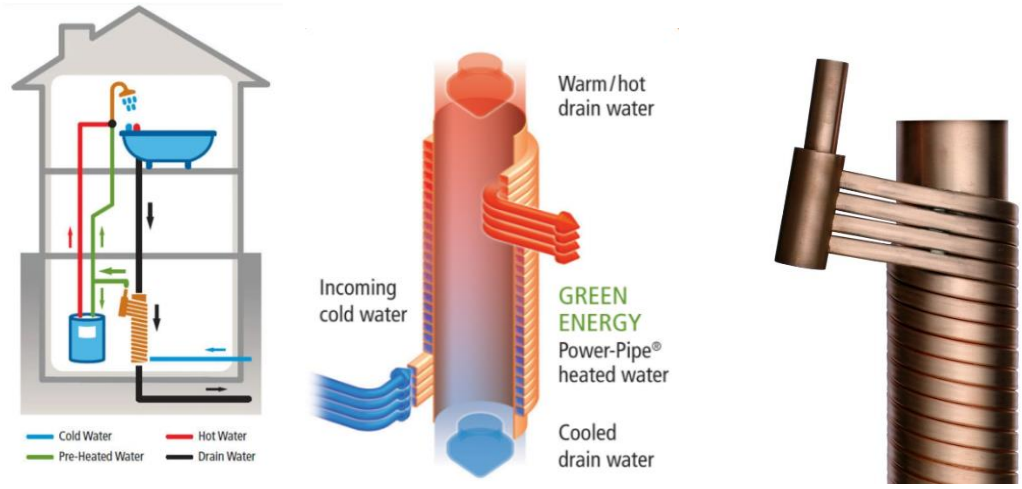


Figure 5: Working mechanism of coiled pipe (Power-Pipe®) (RenewABILITY Energy Inc. (REI), 2020)

Another option for on-demand DWHRS is a field-tube heat exchanger (e.g. Showersave, VX-pipe) in the form of a shower drain which can be seen in Figure 6. The hot drain water is collected and a part of its thermal energy is transferred to the water in the outside part of a field-tube heat exchanger. This system recovers the heat directly at the drain and can decrease the size of the system significantly if only the shower flow is to be heated. It can also be used to heat the central heating flow (Grundén and Grischek, 2016). A comparison between HeatSnagger, HX-drain and VX-pipe has been calculated in the work by Kleven, (2012). Energy used per shower was estimated to be 1.715 kW. After applying the heat recovery system, the reduction in energy per shower was calculated to be from 15.6% for HeatSnagger, 35% for HX-drain to 42.7% for VX-pipe.

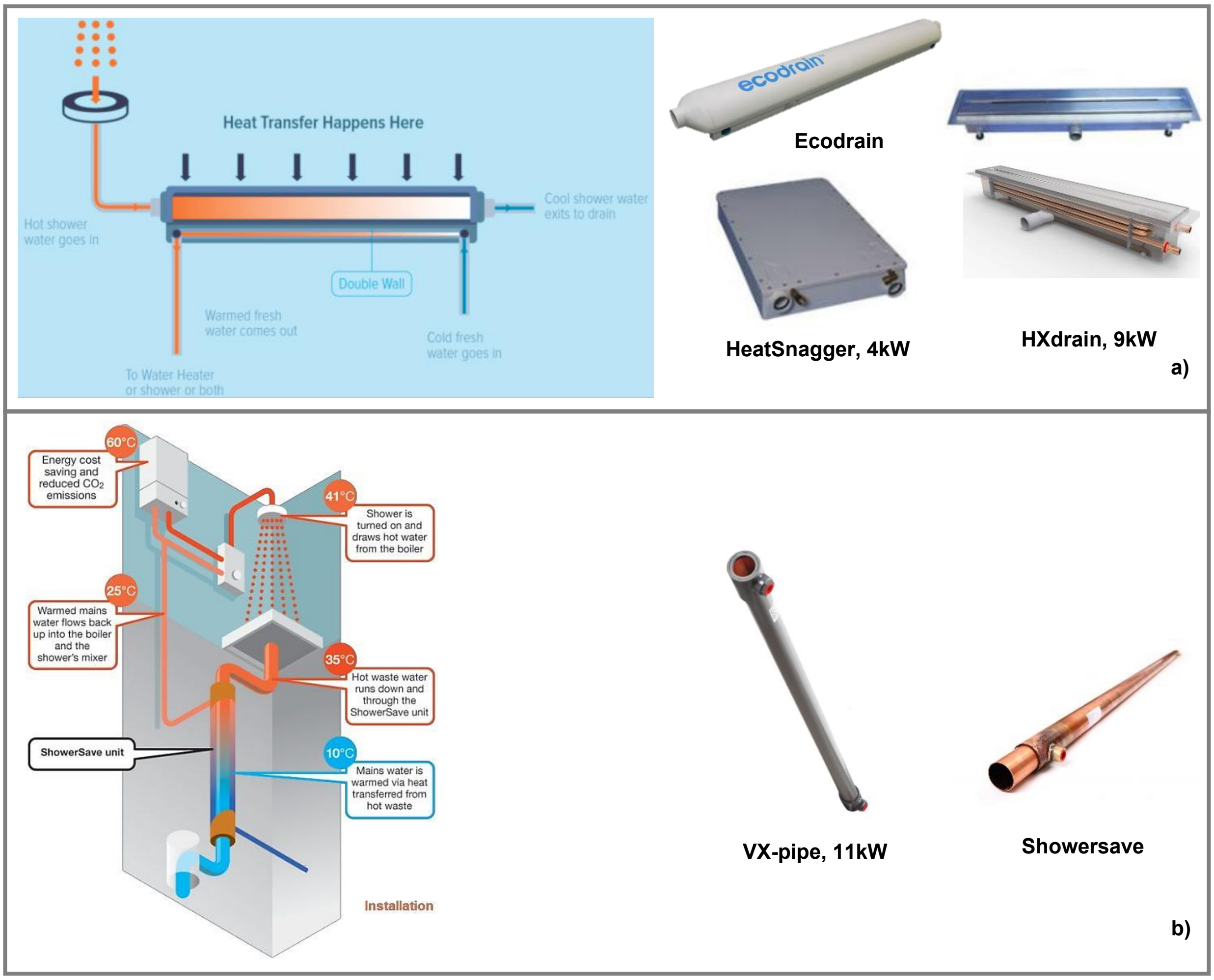


Figure 6: a) Working mechanism of horizontal shower drain heat exchanger (left) (Ecodrain, 2020) and some small scale commercially available heat recovery system products (right) (Ecodrain, 2020; HeatSnagger, 2020). b) Working mechanism of vertical shower drain heat exchanger (left) (Showersave, 2020) and some small scale commercially available heat recovery system products (right) (HeatSnagger, 2020; Showersave, 2020).

Flat plate (see Figure 7) DWHR systems (Ecodrain, Hotshot) are also used and installed under the showers. However, PHE cannot withstand large pressures and temperatures unless it is brazed and therefore does not contain any gaskets. The physical limits for common PHE gaskets are pressures exceeding 20.7 bar and temperatures exceeding 149°C (Kuppan, 2000). The operation conditions for DWHRS (5.5 bar, 49°C) are well below these limitations.

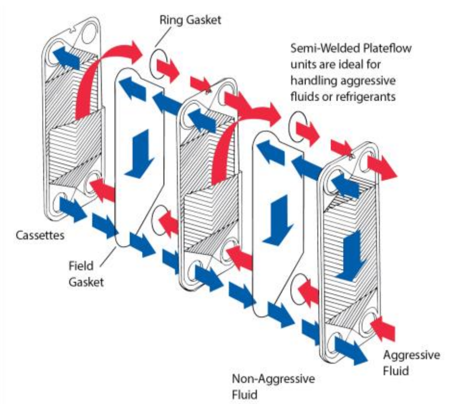


Figure 7: Flat plate DWHR systems (Houkinc, 2020)

5.6 Centralized greywater storage type heat recovery system

In this master thesis the aim was to question whether collecting greywater in a storage and recovering its heat is a feasible idea. In the storage type heat exchangers described by Cooperman and Dieckmann (2011), the warm drain water collects and flows through the tank, heating the clean water flowing in the coiled pipe for future use (Figure 8). The literature on centralised storage heat recovery systems in residential buildings is very scarce and a complete home waste heat recovery system does not exist in the market. This provides great potential to invest in this field and develop a centralised waste heat recovery system that is included in the construction of residential buildings. However, a few studies on this type of heat recovery systems have been carried out and confirmed its feasibility. For example, Torras *et al.* (2016) used both numerical and experimental tools to design, study and test the performance of DWHR, focusing on the analysis of a specific drain water heat recovery storage-type based on a cylindrical tank with an internal coiled pipe (Figure 8).

Figure 8: Storage type heat recovery system (Torras *et al.*, 2016)

The maximum heat recovery of storage type DWHR built by Torras *et al.* (2016) was reached at the lowest flow rates (3 L/min) for the two different in-tank temperatures. The DWHR storage had the capacity to recover from 34% to 60% of the energy available in the drain water for the investigated flow rates. A heat loss test was also conducted. There were no significant temperature gradients in the radial direction. A 50% reduction in stored energy was observed at 24 h, which reveals its limitations for long-term storage applicability. Next, the report from Gavilán *et al.* (2015) investigated the potential for saving energy and money with greywater. The building studied had 23 apartments on 5 different floors and a total living area of 400 m^2 in each floor. 60% of total water used in the case building was hot water. In this report two different solutions to save energy were tested, the first one was to use a heat exchanger only in the shower drains which resulted in saving up to 7.045 MWh or using a centralized heat exchanger saving up to 23.16 MWh. It was concluded that the best heat recovery system to install was a centralized heat exchanger system, since it was found to save more energy and the investment costs are lower than applying heat exchanger in each shower. This gives enough reaffirmations for the selected design of storage type heat exchanger in this master thesis for further work that combines greywater treatment and heat recovery in one technological system.

6 GREYWATER TREATMENT TECHNOLOGIES

Since greywater is normally the ‘light’ version of sanitary wastewater, i.e. it contains everything that wastewater does, just in smaller concentrations, similar technologies to those treating wastewater are applied. And although greywater is relatively less polluted than blackwater or sanitary wastewater, it needs some treatment before its reuse. In fact, none of the untreated greywater characteristics fits to existing reuse guidelines and standards (Ghaitidak and Yadav, 2013). In general, we can distinguish between the two groups of technologies: intensive and extensive.

The first group is based on intensifying the treatment processes by introducing more energy in the treatment system. Typical representative is conventional activated sludge, where energy is used for aeration and returning or keeping the sludge in reactor for more efficient process. Because of that, these technologies require relatively small footprint and added energy for achieving the treatment goals. More advanced technologies, that currently exist for on-site greywater treatment include SBR, fixed film reactors (Pidou *et al.,* 2007; Li *et al.,* 2009) and highly automated and energy-intensive systems that include biological, chemical and physical treatment mechanisms (Friedler *et al.,* 2005; Li *et al.,* 2009; Winward *et al.,* 2008a) such as MBR or rotating biological contactors (RBC), and biological aerated filters (BAF) which showed high removal efficiencies for most of the water quality parameters (Surendran and Wheatley 1998; Jefferson *et al.* 2001; Pidou *et al.* 2007).

In contrast the second group, i.e. extensive technologies, are based on employing natural processes as much as possible, without using energy for their intensification. These technologies use less energy but have larger footprint compared to the intensive ones for achieving treatment goals. They are also called nature-based solutions (NBS). One of the most typical representatives of NBS for greywater treatment are soil and plant based systems, i.e. treatment wetlands. The green wall technology implemented in this thesis represent a version of a treatment wetland.

6.1 Nature-based solutions (NBS) in wastewater treatment

Humanity has already exceeded planetary boundaries regarding changes on global phosphorus and nitrogen cycle (Rockström *et al.*, 2009). Therefore, a major challenge needed for the 21st century is transitioning from grey towards green infrastructure in order to reduce the environmental hazards generated by climate change and rapid urbanisation, to reconcile human development and the preservation of natural capital by restoring ecological flows and increasing resilience in cities (Davies and Lafortezza, 2019; Francis and Lorimer, 2011; Frantzeskaki, 2019). The application of appropriate technologies based on natural processes that can close the nutrient-water loop on all levels of human activities is needed (Lundholm *et al.,* 2015). These innovative design approach technologies go under a name called nature-based solutions (NBS) (Blok and Gremmen, 2016; Garcia-Holguera *et al.,* 2015). NBS should mimic nature without manipulating nature process. “Mimicking nature” is performed with the aim of simplifying undisturbed mutual hydrology-biota regulation to the greatest extent possible. Contrary, “Nature manipulation” means introducing external agents into the local environment, such as exotic species (Krauze and Wagner, 2019). These green technologies work on the concept that proposes the recovering and reuse of nutrients from nutrient rich water which also includes wastewaters, giving them a new application. Natural wastewater treatment systems have been proven to be efficient, cost effective, and user friendly in many studies. Technologies such as treatment wetlands (TWs), sub-soil filtration, storage and bank filtration, have shown promising capabilities in the treatment of domestic and industrial wastewater (e.g. Vymazal 2014). Another great potential for reconciliation ecology in urban areas are green walls in combination with treatment wetlands, which can implement sustainable concepts into natural building design (Ragheb *et al.,* 2016; Cheng *et al.,* 2010; Tilley *et al.,* 2012).

6.2 Soil/substrate and plant based treatment mechanisms

Treatment wetlands (TWs) as most typical representative of substrate and plant based wastewater treatment are environmentally friendly and passive technologies (Avery *et al.,* 2007), which contribute to sustainability and have become popular because of the 'green' image, robustness and the low operating costs (Castellar da Cunha, 2018; Vymazal, 2005). There are two main types of system, namely free water surface flow (Figure 9) and subsurface flow (Figure 10) (Kraiem *et al.,* 2019; Bang *et al.,* 2019; Vymazal and Březinová, 2015).

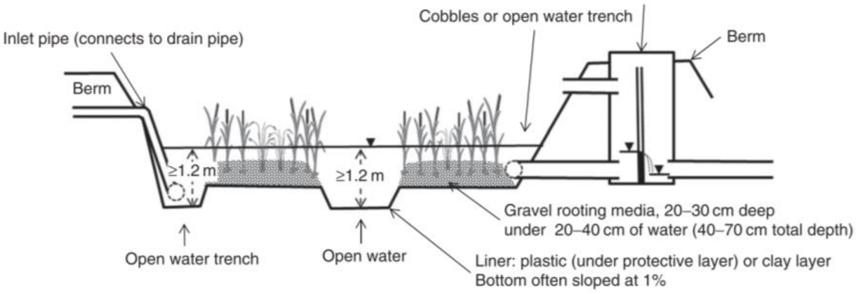


Figure 9: Free water surface wetland. View is a section along the bed length (Fitch, 2014).

Subsurface flow has been preferred in the experiment of this thesis, mainly because this type of flow reduces the risks associated on human and wildlife contact with pathogens present in the wastewater, since wastewater is not exposed during the treatment process (Kadlec and Wallace, 2000). Subsurface flow wetlands (see Figure 10) may be designed as horizontal or vertical flow. In vertical flow, wastewater is fed on the whole surface area through a distribution system. The water then passes the filter media in mostly vertical path and is collected by perforated pipes at the bottom (Yalcuk and Ugurlu, 2009). The water is fed intermittently in order to ensure the aeration of the bed between batch cycles. Therefore, the filter media goes through saturated and unsaturated phases, along with different phases of aerobic and anaerobic conditions. (Vymazal and Březinová, 2015; Huang *et al.,* 2016). Whereas TWs based on a subsurface horizontal flow are usually fed continuously. The wastewater flows through the filter bed and under the surface following a horizontal path until it reaches the outlet zone (Vymazal and Březinová, 2015; Kadlec and Wallace, 2000).

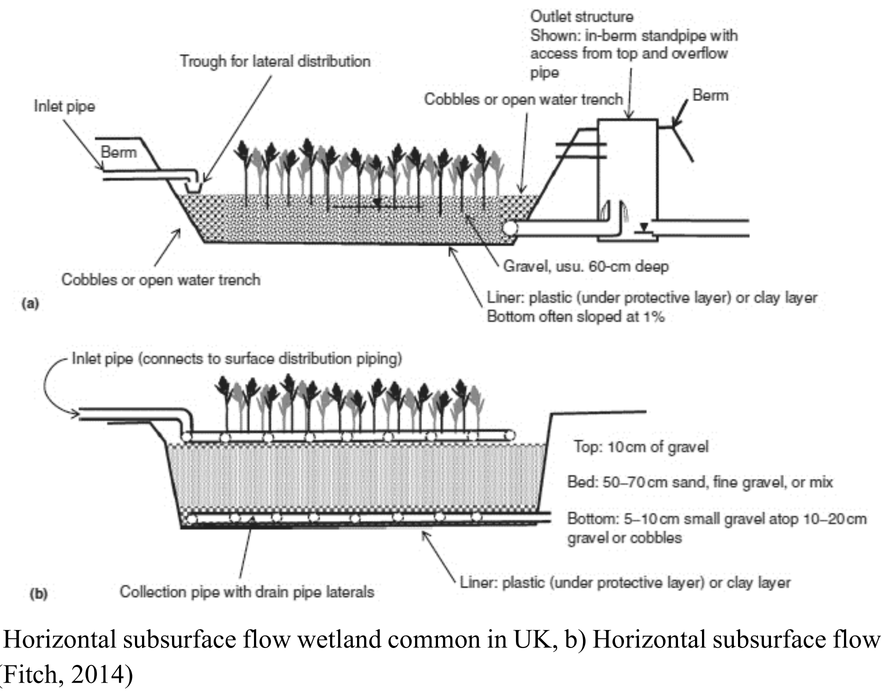


Figure 10: a) Horizontal subsurface flow wetland common in UK, b) Horizontal subsurface flow wetland common in Germany (Fitch, 2014)

Multistage systems, as the combination of horizontal and vertical flow (VF-HF) are probably the most widely used, where the combination of different flows can ensure different redox conditions and thus, enhance the removal of organic matter and nitrification and denitrification process (Vymazal, 2013; Vymazal and Březinová, 2015; Kadlec and Wallace, 2000; Bang *et al.,* 2019).The removal processes of total solid soluble, organic matter and nitrification in hybrid systems are performed in the vertical flow stage, while in the horizontal flow stage denitrification process are favoured by anoxic and anaerobic conditions, if the organic carbon remaining from vertical flow stage is enough to complete the removal of nitrogen (Bang *et al.,* 2019).

Removal of contaminants in wetlands occurs by a variety of treatment mechanisms such as (1) chemical (oxidation, reduction), (2) physical (sedimentation, adsorption, filtration, precipitation) and where applicable (3) biological processes (biodegradation) (Dierberg *et al.,* 2005; da Castellar *et al.,* 2018). These mechanisms are caried out by wetland plants, soils, and associated microorganisms. The removal mechanisms in wetlands are similar regardless of design: solids are removed in the substrate by filtration as well as some settling. The filter media acts as a fixed surface upon which bacteria can attach and a base for the vegetation. The top layer is planted and the vegetation is allowed to develop deep, wide roots, which permeate the filter media and provide habitat for microorganisms.

BOD is decreased as organics are consumed by microbes and ammonia is microbially oxidized near the water surface, and the resulting nitrate can be removed in anoxic metabolism deeper in the wetland. Phosphorus is not biologically removed in wetlands, but some sediment materials may sorb phosphate (Fitch, 2014). The oxygen is transferred in the upper area of the soil and in small amount through vegetation to the root zone so that dense aerobic microbial populations can colonize the area, absorb and degrade the nutrients and organic material. Somewhat surprisingly, plants have a minimal direct impact on pollutant removal. However, they provide surfaces for microbial attachment, insulate and shade the water surface, and affect redox potential in the wetland sediment (Fitch, 2014).

Several studies have been proving that TWs have a great potential to treat greywater (Li *et al.*, 2009; Avery *et al.,* 2007; Comino *et al.,* 2013; Winward *et al.,* 2007). TWs have high potential to remove a wide range of pollutants such as pesticides (Vymazal and Březinová, 2015), pathogens (Díaz *et al.,* 2010; Gruyer *et al.,* 2013), nutrients (especially nitrates and phosphorus) (Vymazal, 2007; Vymazal, 2013; Gagnon *et al.,* 2010), BOD and suspended solids; metals, including cadmium, chromium, iron, lead, manganese, selenium, zinc and toxic organics from wastewater (United States Environmental Protection Agency 2000). These systems were reported around the world to ensure sufficient treatment and can meet most of the standards for reuse about pH, BOD and TSS and if designed with HRT of 3–5 days, single-pass TWs can generally meet restricted reuse chemical/physical standards (Arden and Ma, 2018). However, in some cases post-treatment of the TWs effluent to remove As, EC, E. coli and Helminth eggs might be a requested to make it fit for various reuse applications (Ghaitidak and Yadav, 2013). As the treatment wetlands age, the rate of organic removal increases. Vymazal (2005), Frazer-Williams (2008) and Picard *et al.,* (2005) reported that TWs achieved high BOD removal rates of >90%, and removals of >98% for total coliform, faecal enterococci, COD, and suspended solids. Total nitrogen and phosphorus removal can be as high as 98– 99%, respectively (Picard *et al.,* 2005). Surfactant removal of TWs was shown by Gross *et al.* (2007) to be generally good. According to the literature research of TWs, they were recognised as having a reputation of a reliable and sustainable wastewater treatment system. Therefore, their basic design and working mechanisms were adopted at development of a green wall pilot system in this thesis.

6.3 Removal of organic matter and nutrients

To better understand the greywater treatment in wetlands and green walls it is necessary to study the decomposition of nutrients in wastewater. The experiment in this master thesis was focused on removal of organic matter and partially nutrients. The greywater treatment in the green wall was caried out by biological treatment processes, meaning microorganisms being responsible for pollutant removal. Biological wastewater treatment processes linked to microorganisms and oxygen conditions are collected in Table 12.

Table 12: Summary of biological wastewater treatment processes

Pollutant	Process	Organisms	Conditions	Products
Organic matter, C_{org}	Aerobic oxidation	Heterotrophs	Aerobic	CO_2, new biomass
NH_4^+	Nitrification	Autotrophs	Aerobic	NO_3^-, new biomass
NO_3^- and C_{org}	Denitrification or anaerobic oxidation of C_{org}	Heterotrophs (respiration with NO_3^-)	Anoxic	N_2
P_{tot}	Release of PO_4^{3-}	Heterotrophs	Anaerobic	PO_4^{3-}
PO_4^{3-}	Luxury uptake by PAO Dephosphatization	PAO (phosphorus accumulating organisms)	Aerobic and anaerobic	Biomass loaded with PO_4^{3-}

6.3.1 COD and BOD removal

Biological and chemical oxygen demand (BOD and COD) are parameters that are used to measure organic pollution in water in terms of the amount of oxygen required to oxidise all oxidizable compounds in water. The BOD fraction represents the fraction of organic matter in water which can be readily metabolised by microorganisms in the water, while the COD is the fraction which can be chemically oxidised. The BOD and COD concentration of greywater depends largely on the amount of water used and the household cleaning products. The BOD/COD ratio is an indicator of greywater biodegradability. Greywater is regarded as medium biodegradable with a BOD/COD ratio of 2.9–3.6 (Morel and Diener, 2006). Note that in municipal wastewater this ratio can be up to from 0.4 to 0.7 (Henze, 2008). Mechanisms of pollutant – Organic carbon – removal in treatment wetlands is presented in the Figure 11. Reduction of biological oxygen demand (BOD) or chemical oxygen demand (COD) in a wetland can be primarily attributed to aerobic heterotrophic microorganisms. The result of aerobic respiration by heterotrophs is new cell mass, water, and carbon dioxide. The supply of oxygen in a wetland is largely through the water surface, with some small amount of oxygen provided into the sediment by diffusion through plant roots (Fitch, 2014). BOD is primarily being degraded in the upper, non-saturated filter media layer (which is more aerobic) and that limited organic material is transported into the lower saturated zone for denitrification. This is plausible, given that microbial density and activity are maximised in the first 5-10 cm (aerobic zone) (Faugreen walletter *et al.,* 2009) with depth depending on BOD loading and porosity of the sediment material.

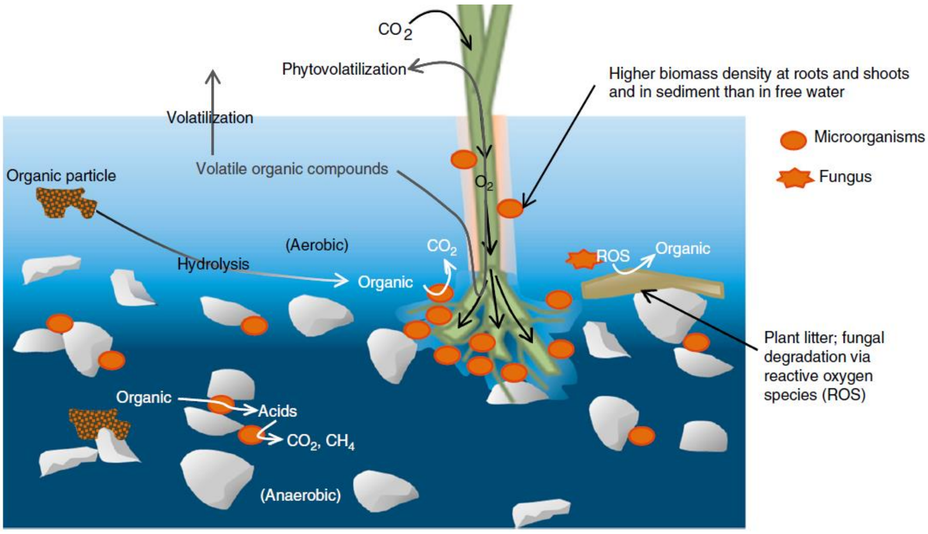


Figure 11: Mechanisms of pollutant – Organic carbon – removal in treatment wetlands (Fitch, 2014)

Treating high levels of BOD aerobically requires large amounts of oxygen, but oxygen diffusion rates into the sediment are quite low. Therefore, a large fraction of organic removal in subsurface wetlands with horizontal flow is attributed to anaerobic microbes (Fitch, 2014) in saturated zones, where anaerobic digestion by sulphate-reducing microorganisms are associated with odours due to H_2S production (Jeppesen, 1996). Elmitwalli and Otterpohl (2007) reported a total anaerobic biodegradability of 74% for greywater from Germany. This value was similar to the 70% biodegradability reported for low-strength dormitory greywater from Jordan (Ghunmi, 2009). A study from Hernández *et al.* (2011) reported that the high anaerobic biodegradability of 70%, indicates the possibility of recovering COD as methane, however the low hydrolysis constant of 0.02 d^{-1}, may limit the application of anaerobic greywater treatment. Surfactants were found at high concentrations, especially anionics (41.1 mg/L). At this concentration, anionics have the potential to inhibit anaerobic processes.

Aerobic treatment, therefore, may be more suitable for greywater treatment because anionics do not present toxicity for aerobic processes and are even biodegraded to a large extent and because of lowered odours due to no H_2S production. Furthermore, a BOD:COD ratio gives an indication of treatability of a wastewater. The low BOD:COD ratios (<0.3) give an indication that the influent greywater is less treatable by biological means than typical domestic wastewater and there is a need for acclimated microorganisms (Metcalf and Eddy Inc. 2003). Past studies that have characterized individual greywater streams suggested the potential for nutrient deficiency (e.g. high C:N ratio) may occur, particularly if kitchen water is excluded (De Gisi *et al.,* 2016; Jefferson *et al.,* 2004; Li *et al.,* 2009), consequently reducing the efficiency of biological treatment processes. For example, Hang *et al.* (2016) recommended C:N ratios of at least 4:5 and 1.8:3.0 for TWs and bioreactor, respectively.

Next, a COD:N:P ratio of 100:20:1 is required for aerobic treatment (Metcalf, 1995) and a ratio of 350:5:1 is required for anaerobic treatment. In greywater, this ratio is 100:3.5 ± 1.3:1.6 ± 0.7, which indicates a nitrogen deficiency for aerobic treatment, but not for anaerobic treatment. Usually only about 3% of the nitrogen from household wastewater is discharged with greywater, as about 87% is in urine and 10% in faeces (blackwater) (Otterpohl, 2002).

6.3.2 Nitrogen removal

Proteins make up more than half the dry mass of cells. As protein and other nitrogen-containing cell components are degraded, the nitrogen may be released if the degrader organism has a low nitrogen requirement. Thus, organic nitrogen may become a source of ammonia or nitrate. Generally, organic nitrogen is most likely to be assimilated into biomass (Fitch, 2014). Drawing from studies of other vegetated water treatment systems, e.g., wetland systems, it is hypothesised that N retention mechanisms are possibly restricted to denitrification, plant assimilation and to a lesser extent ammonium ($NH4^+$) adsorption and organic N burial (Vymazal *et al.,* 2007). Assimilation was found to be the dominant nitrate (NO_3^-) removal pathway in comparison to denitrification in laboratory scale stormwater biofilters (Payne *et al.* 2014). Mechanisms of pollutant – Nitrogen– removal in treatment wetlands is illustrated in Figure 12. In vertical flow subsurface wastewater wetlands receiving higher N loads, denitrification is similarly low while plant harvesting and accumulation of organic matter in soil can make slightly greater contributions towards removal (Meuleman *et al.,* 2003). In contrast, coupled nitrification-denitrification has mostly been found to be the dominant process removing N in sub surface horizontal flow wastewater wetland systems (e.g., Maltais *et al.,* 2009), covered by either anaerobic bacteria when oxygen is absent or a third class of bacteria, facultative anaerobic bacteria, which utilize free oxygen when it is available and use other substances as electron receptors (oxidants) when molecular oxygen is not available (anoxic conditions). Common oxygen substitutes in water are nitrate ion and sulphate ion (Manahan, 2010).

Moreover, the main form of N in greywater is typically dissolved organic nitrogen (DON) such as urea, dissolved free amino acids, proteins, nucleic acids, amino sugars, and humic. DON can be converted to more bioavailable forms, e.g., to NO_3^-, which is highly mobile and can easily leach and have more damaging consequences in the environment (e.g., eutrophication, aquatic and biodiversity loss as discussed earlier) if a reliable process to remove this contaminant is absent within the treatment system (Eriksson *et al.,* 2002; Antia *et al.*, 1991).

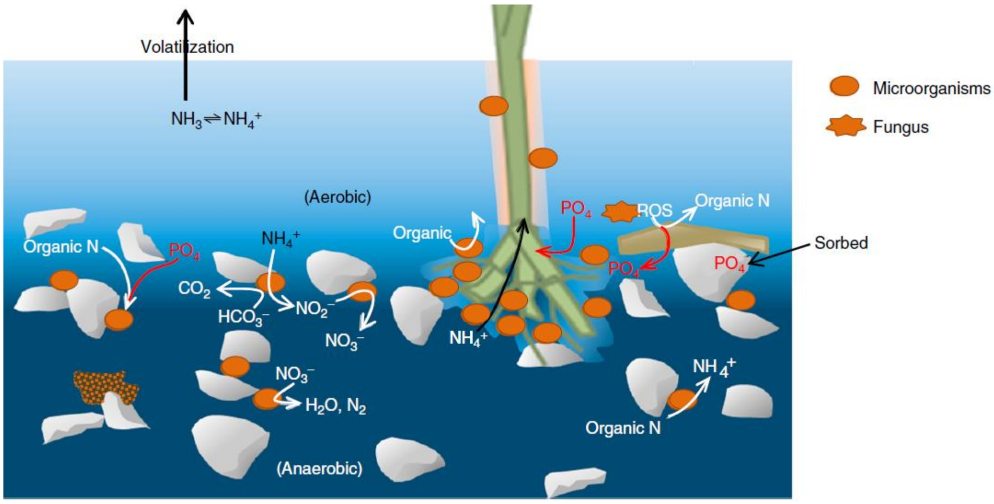


Figure 12: Mechanisms of pollutant – Nitrogen and Phosphorous – removal in treatment wetlands (Fitch, 2014)

Ammonia vs. Ammonium

Ammonia and ammonium are different forms of nitrogen. The major factor that determines the proportion of ammonia to ammonium in water is pH. When the pH is low, the reaction is driven to the right, and when the pH is high, the reaction is driven to the left. The activity of the ammonia is also influenced by ionic strength and temperature. Un-ionized NH_3 can be harmful to aquatic organisms, while ionized ammonium is basically harmless (HACH, 2019). The chemical equation that drives the relationship between ammonia and ammonium is:

$$NH3 + H_2O \leftrightarrow NH4^+ + OH^- \quad (1)$$

Nitrification (ammonia and nitrite oxidation)

In nature, nitrification is catalysed by two groups of bacteria, Ammonia-oxidizing bacteria (AOB: *Nitrosomonas spp., Nitrosococcus spp., Nitrosospira spp., Nitrosolobus spp.,* and *Nitrosovibrio spp.*) that catabolize ammonia to nitrite; and Nitrite-oxidizing bacteria (NOB: *Nitrococcus spp., Nitrospira spp., Nitrobacter spp., and Nitrospina spp*.) which transform nitrite into nitrate (Hagopian and Riley, 1998).

Nitrosomonas bacteria bring about the transition of ammonia to nitrite,

$$NH_4^+ + 2O_2\ (nitrifying\ bacteria) \rightarrow NO_3^- + 2H^+ + H_2O \tag{2}$$

$$NH_3 + \frac{3}{2}O_2 \rightarrow H^+ + NO_2^- + H_2O \tag{3}$$

Nitrobacter mediates the oxidation of nitrite to nitrate:

$$NO_2^- + \frac{1}{2}O_2 \rightarrow NO_3^- \tag{4}$$

Both highly specialized types of bacteria are obligate aerobes; that is, they function only in the presence of molecular O_2 (Manahan, 2010). Therefore, nitrification will occur only near the atmosphere, in the water column and surface of the wetland sediment, and possibly near the roots of wetland plants (Fitch, 2014). Although nitrifiers are autotrophic, they are not photosynthetic and require oxygen to consume ammonia (Fitch, 2014).

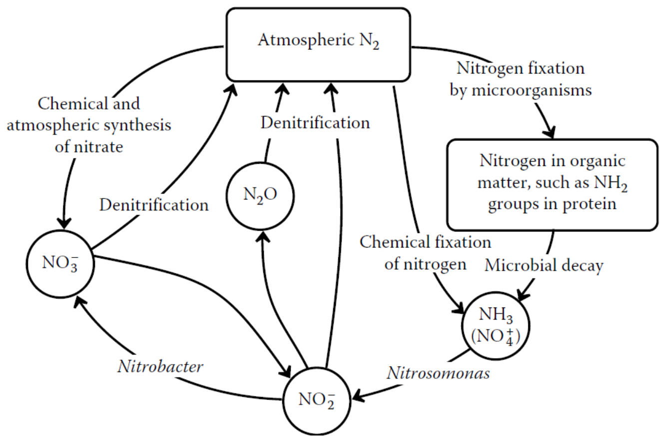


Figure 13: The nitrogen cycle (Manahan, 2010)

Ammonia oxidation occurs optimally between pH 7.5 and 8.0 and temperatures between 25 and 30 °C (Prosser, 1989). AOB have low growth rates and yields because of the small energy gain from the oxidation of ammonia and large energy investment needed to reduce inorganic carbon, resulting in generation times varying from 8 h to several days (Prosser, 1986, 1989). In addition to biological use, ammonia may evaporate into the atmosphere (Fitch, 2014). Development of nitrifiers in culture water can be accelerated by inoculating new water with bacteria from established nitrifying populations or by adding a commercially available source of bacteria. This is especially important when sufficient carbon is available, then faster growing heterotrophic bacteria outcompete nitrifying bacteria for ammonia (Samocha *et al.,* 2019).

Nitrate reduction

In the absence of free oxygen (anoxic state), nitrate may be used by some bacteria as an alternate electron acceptor. Nitrogen is an essential component of protein, and any organism that utilizes nitrogen from nitrate for the synthesis of protein must first reduce the nitrogen by conversion of +V nitrate to the −III oxidation state (ammoniacal form). This microbially mediated reaction is termed nitrate assimilation. Nitrate ion is a good electron receptor in the absence of O_2 (Manahan, 2010). One of the factors limiting the use of nitrate ion in this function is its generally low concentration in most waters.

$$\frac{1}{2}NO_3^- + \frac{1}{4}\{CH_2O\} \rightarrow \frac{1}{2}NO_2^- + \frac{1}{4}H_2O + \frac{1}{4}CO_2 \qquad (5)$$

Furthermore, nitrite, NO_2^-, is relatively toxic and tends to inhibit the growth of many bacteria after building up to a certain level. Sodium nitrate salt has been used as a "first-aid" treatment in sewage lagoons that have become oxygen deficient. It provides an emergency source of oxygen to re-establish normal bacterial growth (Manahan, 2010).

Denitrification (reduction of nitrate)

The second step is the reduction of nitrate to nitrogen gas. An important special case of nitrate reduction is denitrification, in which the reduced nitrogen product is a nitrogen-containing gas, usually N_2.

$$\frac{1}{5}NO_3^- + \frac{1}{4}\{CH_2O\} + \frac{1}{5}H^+ \rightarrow \frac{1}{10}N_2 + \frac{1}{4}CO_2 + \frac{7}{20}H_2O \qquad (6)$$

This reaction is also bacterially catalysed and requires a carbon source and a reducing agent such as methanol, CH_3OH (Manahan, 2010).

$$6NO_3^- + 5CH_3OH + 6H^+(denitrifying\ bacteria) \rightarrow 3N_2(g) + 5CO_2 + 13H_2O \qquad (7)$$

Denitrification is an important process in nature. It is the mechanism by which fixed nitrogen is returned to the atmosphere. Denitrification is also used in advanced water treatment for the removal of nutrient nitrogen. Most wastewater treatment processes utilize chemoheterotrophic denitrification: Organic substrates are required as electron donors and carbon sources. The organic carbon source can be supplied by either an external carbon source such as methanol, or an internal carbon source by using the influent BOD in the wastewater, or organic carbon obtained by endogenous decay of biomass (the breakdown of cellular organic matter) (Ergas*et al.,* 2014). A carbon source (woodchips, coco fibre, coir) is usually provided in the saturated zone of biofilter systems receiving low organic polluted waters (e.g. stormwater) to facilitate NOx removal via denitrification. A recent study by Fowdar *et al.* (2017) suggested that light greywater may possess enough biodegradable organics to promote satisfactory denitrification on its own under anoxic conditions with dissolved oxygen (DO) concentration less than 0.2 mg/L (Seitzinger *et al.* 2006). Denitrifying bacteria are a part of the N cycle and consist of sending the N back into the atmosphere. Most denitrifying bacteria are facultative heterotrophic anaerobes and thus denitrification progresses well in the absence of appreciable DO (Fitch, 2014) Because nitrogen gas is a nontoxic volatile substance that does not inhibit microbial growth, and since nitrate ion is a very efficient electron acceptor, denitrification allows the extensive growth of bacteria under anoxic conditions where DO concentrations are low and NO_3− is present, they will couple the oxidation of organic carbon compounds to CO_2 with reduction of NO_3− to N_2 gas. However, at higher DO concentrations, denitrifiers preferentially utilize the more thermodynamically favourable O_2 as an electron acceptor, and denitrification is inhibited (Ergas *et al.,* 2014). Loss of nitrogen to the atmosphere may also occur through the formation of N_2O and NO by bacterial action on nitrate and nitrite catalysed by the action of several types of bacteria. Production of N_2O relative to N_2 is enhanced during denitrification in soils by increased concentrations of NO_3^-, NO_2^-, and O_2 (Fitch, 2014).

6.3.3 Phosphorous removal

Phosphorous is removed from aquatic environment by (1) assimilation of organisms (algae, bacteria, plants), and (2) precipitation of minerals (e.g., Ca, Fe, Al) with sorption being the general removal mechanism (see Figure 12 and Figure 14).

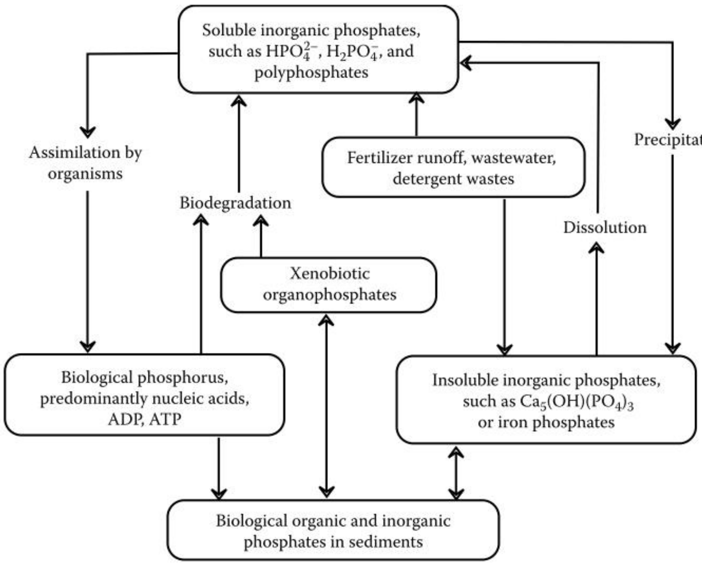


Figure 14: The phosphorous cycle (Manahan, 2010)

Phosphorous assimilation by organisms

Cells contain some amount of phosphorus (P), largely as phosphate in some form, but net P removal by cells is low. The amount of P stored in cells varies among organisms and depends on growth conditions. On the low side, cells may only contain 0.03% by weight P (Cotner and Hall, 2011), and on the high side, phosphorus accumulating organisms may have 20% or more of the dry weight as P. Thus, as biomass grows, some P is removed from water into cells (Fitch, 2014).

Firstly, phosphorous can be assimilated by algae. In fact, one g of P released into water bodies promotes the growth of up to 100 g of algae, enhancing eutrophication of the surface waters (Karczmarczyk *et al.,* 2014). As cells senesce and die, though, P is rereleased into the water.

Secondly, phosphorous can be assimilated by plants. Plants require macronutrients (nitrates and phosphorus) for different physiobiological processes (Masi *et al.,* 2016). Their high growing activity needs energy supplying under the form of adenosine triphosphate (ATP). To produce ATP, they take phosphorus in their environment mainly by small roots (Föhse et al, 1991). Deposition of plant litter does not significantly store P, as the litter is almost exclusively carbohydrate. As plant litter becomes new sediment, it sorbs phosphorus. The growth or accretion of this sediment does remove P, with estimates of the annual areal rate being 0.06–3 g P m^{-2} and for one wetland almost 14 g P m^{-2} yr^{-1} reported (Kadlec and Wallace,2009). Without removal of biomass, P removal by cells in biofilters should be considered to be negligible (Fitch, 2014).

Secondly, phosphorous can be assimilated by bacteria under certain conditions, namely by alternating anaerobic and aerobic conditions (see Figure 15). *Acinetobacter* and *Pseudomonas* (Phosphorous accumulating organism, PAO) are added into the BOD rich substrate under anaerobic condition.

Fermentation products (volatile fatty acids (VFA)) from facultative bacteria are assimilated in the cells of PAO. Under anaerobic conditions polyhydroxybutyrate (PHB) is synthesized. To store the organic, PAO use the energy from poly-P which decomposes and releases ortho-P (PO_4). Next, under aerobic conditions, stored organics are a source of energy (oxidation of PHB) for PAO, the energy is used for creating poly-P with luxury uptake of ortho-P (PO_4). Oxidation of PHB accelerates growth of new PAO cells with high storage of poly-P. To remove phosphorous from the effluent, treated wastewater is sent to clarifier that removes sludge with PAO, which can be used as a fertilizer.

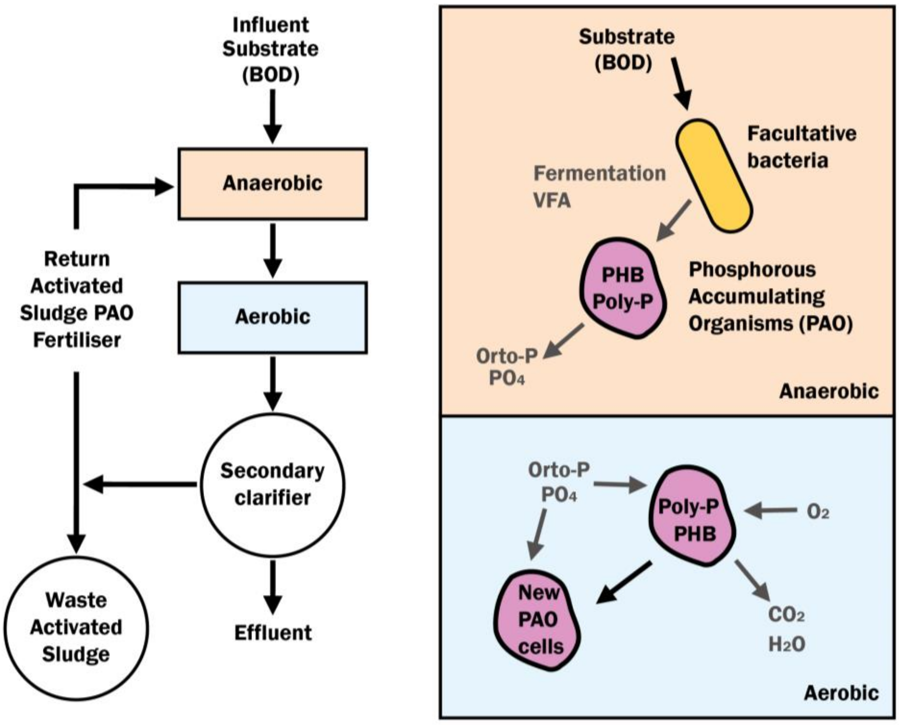


Figure 15: Phosphorous assimilation by two bacteria (PAO): *Acinetobacter* and *Pseudomonas* under aerobic and anaerobic conditions. Abbreviations: Phosphorous accumulating organism (PAO); Polyhydroxybutyrate (PHB); Volatile fatty acids (VFA) (Minnesota Pollution Control Agency, 2006; EPA 1987).

Precipitation, sorption of phosphorous

If a wetland is to be used for P removal, sorption to clay or other positively charged surfaces is the general removal mechanism. The P removal likely starts as adsorption, but with some sorbent chemistries, the sorbed P is incorporated into the solid, such that the mechanism might be termed absorptive or precipitative. Regardless of sorbent and mechanisms, eventually break- through will occur and P removal by sorption will stop. Thus, P removal by sorption cannot be sustained without some active treatment process incorporated into wetland operation. Another potential problem with sorption, particularly adsorption, is that varying P concentrations result in periods of low concentration, which will cause release of P back to the water (Fitch, 2014). The most effective sorbents studied to date contain oxides of Ca, Al, or Fe, such as apatite, bauxite, and various limestone forms. Commercial by-products such as slag may also have a high sorption potential. Many fair sorbents retain in the range of 10–50 g P kg^{-1} sorbent (Vohla *et al.,* 2011), but the size of the sorbent particle strongly influences capacity. It is also important to observe the effect of the sorbent on pH, as many sorbents lower or raise pH significantly. At a higher rate of aeration in a relatively hard water, the CO_2 is swept out, the pH rises, and reactions such as the following occur:

$$5Ca^{2+} + 3HPO_4^{2-} + H_2O \rightarrow Ca_5OH(PO_4)_3(s) + 4H^+ \quad (8)$$

Reaction is strongly hydrogen ion-dependent, and an increase in the hydrogen ion concentration drives the equilibrium back to the left. Thus, under anaerobic conditions when the sludge medium becomes more acidic due to higher CO_2 levels, the phosphate returns to solution (Manahan, 2010). Chemically, phosphate is most commonly removed by precipitation. Some common precipitants and their products are shown in Table 13.

Table 13: Chemical precipitants for phosphate and their products (Manahan, 2010)

Precipitant(s)	Product
$Ca(OH)_2$	$Ca_5OH(PO_4)_3$ (hydroxyapatite)
$Ca(OH)_2$ + NaF	$Ca_5F(PO_4)_3$ (fluorapatite)
$Al_2(SO_4)_3$	$AlPO_4$
$FeCl_3$	$FePO_4$
$MgSO_4$	$MgNH_4PO_4$

Precipitation processes are capable of at least 90–95% phosphorus removal at reasonable cost. Lime has the advantages of low cost and ease of regeneration. The efficiency with which phosphorus is removed by lime is not as high as would be predicted by the low solubility of hydroxyapatite, Ca5OH(PO_4)$_3$ (Manahan, 2010). Some of the possible reasons for this are slow precipitation of Ca5OH(PO_4)$_3$, formation of nonsettling colloids; precipitation of calcium as CaCO3 in certain pH ranges, and the fact that phosphate may be present as condensed phosphates (poly- phosphates) which form soluble complexes with calcium ion. Lime, Ca $(OH)_2$, is the chemical most commonly used for phosphorus removal:

$$5Ca(OH)_2 + 3HPO_4^{2-} \rightarrow Ca_5OH(PO_4)_3(s) + 3H_2O + 6OH^- \quad (9)$$

Taking TWs as a reference, it is possible to predict that the precipitation and adsorption of phosphorus are higher under saturated conditions because of the low fluctuation in redox potential (Vymazal, 2007). Therefore, the selection of materials rich in Fe, Mg, Al, Ca and organic carbon and its allocation under anoxic zones could increase phosphorous removal by precipitation and nitrogen removal by microbiological degradation (denitrification) (Castellar Da Cunha *et al.*, 2018). Adsorption onto the filter media and precipitation reactions as the more dominant removal for TP decreases with time as saturation of filter media with phosphorous occurs (Kadlec and Knight, 1996; Fowdar, 2017; Dzakpasu *et al.,* 2015; Tanner, 1996). Therefore, phosphorous removal is time conditioned and possible only until filter media adsorption capacity is fully reached and after that, the filter has to be replaced with a new one.

7 GREEN WALL TECHNOLOGY FOR GREYWATER TREATMENT

Green/Living wall is a green technology that is gaining momentum in the field of sustainable development. Green walls refer to vegetation that grows directly onto a building façade, or to vegetation that is grown in planter boxes at the base of a building using the wall or a separate structural system adjacent or attached to the wall as support (Loh, 2008; Fowdar, *et al.* 2018). They have a small footprint and can markedly help increase the sustainability and liveability of urban cities through their multiple benefits (Perez-Urrestarazu *et al.,* 2015; Loh, 2008). Green wall installation on buildings have many advantages, it can provide carbon sequestration, acoustic comfort, have the ability to create and preserve ecological biodiverse habitats for animals such as birds, bees and insects. improve microclimate, provide considerable energy savings, through the reduction of thermal load on buildings and indoor air temperature, facts which can reduce energy expenditure on air conditioning and help to reduce the urban heat island (UHI) effect issue (Raji *et al.,* 2015, Cameron *et al.,* 2014).

UHI effect is reported to impact public health and comfort, resulting in increased cooling requirements to counteract its effects, consequently becoming a secondary contributor to the heat islands (Moreno-Garcia, 1994). For example, in Saudi Arabia, ventilation and air conditioning accounts for 70% of the electrical energy used in residential buildings (Dawood and Vukovic, 2017). As a solution, vertical greening systems can be efficient at affecting energy demands for heating, during winter, air conditioning and reduction of indoor temperatures during summer by the shading and evapotranspiration of plants and substrate (Raji *et al.,* 2015). For instance, evapotranspiration unaided or in combination with shading can reduce peak summer temperatures by 1–5°C (Laaidi *et al.,* 2012). Results from Coma *et al.* (2014) highlighted a reduction of temperature up to 14 ºC during summer in the external south surface after green walls were used, mainly due shadow effect and consequently, leading to slight reduction in energy consumption. Perini *et al.* (2011a) evidenced that the foliage coverage can create a stagnant air layer increasing the thermal resistance of the building facade up to 0.09 $m^2 \cdot K \cdot W^{-1}$.

Commercial “Ambient” green walls offer multiple benefits to urban environments, but they are major water consumers and not optimal for use in dry climates (Pérez- Urrestarazu *et al.,* 2015), since they require 0.5–20 L/m^2 of potable water per day (DEPI, 2014). Aiming to find a solution, Kew *et al.* (2009) trialled the use of stormwater as an alternative source of water to irrigate green walls, but the irregularity of its generation and subsequent need for storage were found to be significant operational challenges. Some of the green walls are currently watered with recycled greywater to lower the use of drinking water which would be used for watering instead (Hopkins and Goodwin, 2011). Additionally, if green walls were engineered to treat greywater, they could become cost-effective and more widespread. (Rysulova *et al.,* 2017).

Although green walls are becoming increasingly popular, since they provide multiple benefits, they are still to be fully developed as effective wastewater treatment systems. Therefore, it is proposed to develop an efficient and sustainable hybrid system that reuses greywater and undertakes the treatment wetland treatment ability and transform it into smaller area with comparable treatment efficiency (Rysulova, 2017). Unlike the treatment wetlands, which construction demands great land area, the green walls are using blank large surface spaces of building walls and empty facades in urban areas, where little space is available (Marchi *et al.,* 2015). Although this concept has been put forward by a few product developers (e.g. Gunther, 2013), there is a limited number of published studies on design, operations and governing processes in such systems, especially on specific design of linear planter box-horizontal flow green walls used in this thesis. However, these studies have relied on black-box experiments, not examining processes within the systems, or how to enhance them. The role of each green wall element in greywater treatment and pollutant capture, including media, plants and structural design, is still yet to be understand so that optimal treatment systems can be produced (Prodanovic *et al.,* 2017).

7.1 Types of green walls

Literature research reveals a variety of definitions referring to all types of vertical greening systems, causing sometimes confusion and misunderstanding in assigning systems and components (Medl *et al.,* 2017). Commonly applied terms are "green wall system", "vertical greening system", "vertical greenery system" or "green vertical system". Green façades are also known as "façade greening", while "vertical garden" and "living walls" are common nomenclatures for green walls (Medl *et al.,* 2017 and references therein). However, these names can be categorised as either green façades or living walls (Bustami *et al.*, 2018). The categories are based on their construction system. The green walls can be divided into three fundamental types according to the species of the plants, types of growing media and construction method:

1. Ground based – Wall climbing green façades is the very common and traditional green walls method. Ground-based greening method relies on natural ground and refers to green façades. Green façades are further classified according to the location of plants, which can either be placed directly into the soil, or in soil-filled planter boxes (Mir, 2011). Although it is a time-consuming process, climbing plants can cover the building walls naturally. Sometimes they are grown upwards with the help of a trellis or other supporting systems (Wilmers, 1990). Green façades involve climbing plants to cover vertical surfaces and are subdivided into:

a) direct green façades and
b) indirect green façades both options can be planted directly in the ground or in planter boxes and have a life span of more than 50 years (Perini *et al.*, 2013)

2. Wall based - Living wall by contrast, refer to vegetation grown in planter boxes which can be developed into modular systems attached to walls without relying on rooting space at ground level and having mechanised watering. Modular panel systems are popular and they can be in the form of trays, vertical or horizontal felt systems, among others (Bustami *et al.*, 2018). The plants in the wall-based green wall are not directly contacting on the building façade and are not connected to natural ground. They are designed with pre-vegetated panels, vertical modules, felt layers, planter boxes, steel baskets or planted blankets that are fixed vertically to the surface and are used to sustain substrate and plants by allowing their growth without relying on rooting space at ground level (Köhler, 2008; Feng *et al.,* 2014; Kontoleon and Eumorfopoulou, 2010; Perini *et al.,* 2011b). Living walls can be implemented both, indoor and outdoor. It requires more complicated design and planning considerations before a vertical system can come to place. It is also probably the most expensive method (Jonathan, 2003). The Wall based - Green walls can be divided into three types according to the construction method:

a) Continuous green walls are based on a single support structure. A fabric layer (permeable, flexible and root proof screens, also serving as drainage) serves as a growing media (Manso *et al.,* 2015; Dover *et al.,* 2015; Charoenkit and Yiemwattana, 2016).
b) Modular green walls result from the installation of several modular elements in form of pocket-typed planters and panels, together forming the whole greenery (Manso *et al.,* 2015). Each module is designed to hold soil or substrate and is fixed to a structural frame behind (Charoenkit and Yiemwattana, 2016).
c) Linear green walls result from cascading elements, affixed to the wall in a linear way. They are composed by linear planter boxes (e.g. aluminium or plastic (HDPE)), that are applied one above the other and filled with substrate (e.g. soil or mineral granules, (Scharf *et al.* 2012; Pitha *et al.,* 2011; Manso *et al.,* 2015).

While there are multiple commercially available green wall designs (Jim, 2015; Medl *et al.,* 2017), only modular, containerised designs are suitable for greywater treatment, due to their significant media volume, which is crucial for the removal of certain particulate and dissolved pollutants (Prodanovic *et al.,* 2017, 2018). It is hypothesised that the key benefit of the containerised design for water treatment would be its facilitation of complex interactions of plants' root systems within a single container unit. In certain design and operational conditions this can enhance nutrient uptake and distribution within the system, benefiting pollutant removal and plant health (Bais *et al.,* 2006). Therefore, in this thesis linear green wall was recognised as the most suitable system for greywater treatment as it is assumed of being able to undertake the treatment wetland pollutant removal mechanism due to its similar form.

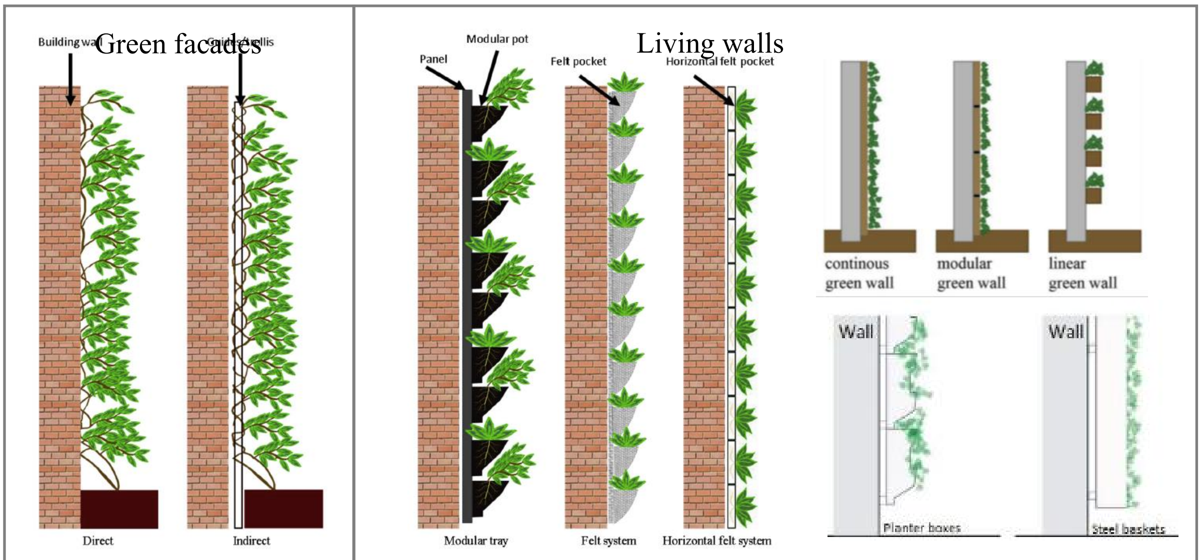


Figure 16: Classification of vertical greening systems. Ground-based green façades (direct and indirect green façade) and Wall-based living walls (continuous, modular, linear/planter box, steel basket, felt system). (Bustami *et al.*, 2018; Safikhani *et al.,* 2014; Hunter *et al.* 2014; Medl *et al.,* 2017; Perini *et al.*, 2011a).

7.2 Design parameters

While planning a greywater treating green wall, some aspects must be considered. Firstly, dimensions and volumes of green wall frame and construction are calculated to fit the daily hydraulic (HL [m3/d]) and biochemical loads (BL [g BOD/d]) and organic load (OL [mg COD/L and mg BOD/L]). Next, it is important to choose a suitable substrate/filter and plants that are appropriate for greywater treatment. Depending on the treatment goals, i.e. if denitrification is required, the substrate must ensure enough carbon availability to complete denitrification requirements. It is important to consider unsaturated and saturated conditions, required for both nitrification and denitrification processes. Lastly, it is important to design a green wall in a way that avoids reduced hydraulic conductivity over time as a result of clogging. Main design parameters and results on pollutant removal provided by green walls wastewater treatments, by different studies were collected in Table 14. The parameters collected from different studies are compared and dicsused with results obtained in this study in the section 9.

Table 14: Main design parameters and results on pollutant removal provided by green walls wastewater treatments, by different studies

		Green wall in Master's Thesis (2021)	**Prodanovic *et al.*, (2020)**	**Zraunig *et al.* (2019)**	**Castellar Da Cunha et al (2018)**	**Fowdar *et al.* (2018)**	**Fowdar *et al.* (2017)**	**Masi *et al.* (2016)**	**Svete *et al.* (2012)**
Type of Greywater	Mixed/ Light, Synthetic/ Sourced	Mixed GW, synthetic	Light GW, synthetic, 1 dosing per day	Light GW, hotel	Light GW, synthetic, recirculating 1x day	Light GW, synthetic, applied 5 times per week	Light GW, synthetic, applied 5 times per week	Light GW, office building	Mixed GW, student dormitorie
Country	/	Slovenia	Australia	Spain.	Spain	Australia	Australia	India	Norway
Duration	Months	4	12	22	9	8	12	9	3
Green wall type	/	Linear containers, Timmer based	1_Pot green wall design (Gro-wall®) and 2_Block green wall design. Drip irrigation system.	Linear containers	Vertical and linear tube	Biofilter columns of 240 mm PVC pipe	Biofilter columns of 240 mm PVC pipe	Matrix of pots. Drip irrigation. Timer based	Single-pass intermittent media filter. Spray nozels. Timer based
VF/HF	/	HF	VF	HF	VF and HF	VF	VF	VF	VF
Dimensions	/	4x Containers: l: 160 cm x h: 26 cm x w: 20cm	1_Each pot 150 mm deep, containing 6 L of media mix, total 450 mm and 18 L media depth, 2_Block green wall: circular PVC pipe, 450 mm long and 230 mm in diameter. Each filled with 18 L of media mix.	l:1.5m ×w:1.5m and h:2.5m 4x Containers: h: 40 cm x w: 35cm	Two modules with VF and HF l: 110 cm x Ø: 20cm.	240 mm PVC pipe, h_1:0.94 m, h_2: 0.8 m	240 mm PVC pipe, h_1:0.94 m, h_2: 0.8 m	12 × 6 matrix of pots (6 pots in a column and 12 pots in a row).	h: 195 cm, w: 35 cm
Indoor/ Outdoor	/	Indoor	Open-air greenhouse	Semi-indoor with an encapsulation in winter	Outdoor	Open air greenhouse	Open air greenhouse	Outdoor: on the front walls of the office	Outdoor

Substrate	/	Perlite, silica sand, coco fibres	1:2 mix of perlite and coco coir.	LECA	Crushed autoclaved aerated concrete (CAAC) and cork	1_ Washed sand, washed sand+carbon, coarse sand, gravel. 2_Washed sand, coarse sand, gravel, pannels	1_ Washed sand, washed sand+carbon, coarse sand, gravel. 2_Washed sand, coarse sand, gravel, pannels	1_Expanded clay (LECA) + sand and 2_LECA + coconut fibres	Lightweight expanded clay
OLR of GW	g COD/m^{2}d	All beds: 68, 33 First bed: 272, 132	n/a	15.9, 21, 34	n/a	n/a	n/a	10	230,
Inflow concentration	mg/L	608, 347 COD mg/L, 133, 99 BOD mg/L	317 COD mg/L 5.1 TN mg/L	158 COD mg/L, 116 BOD mg/L	30 BOD mg/L 10 NH4-N mg/L 10 NO3-N mg/L 10 PO4-P mg/L	1_36 BOD mg/L, 2_50 BOD mg/L	104 BOD mg/L	20–100 COD mg/L, 6–47 BOD mg/L, 3.5 NH4-N mg/L	241 COD mg/L
BOD:COD	/	0.24	n/a	0.7	n/a	n/a	n/a	0.43	n/a
Feeding/ Resting	min/h	continuous and 15 min/ 0.25 h	1x day 4l	4.5 min/ 1h, 6 min/ 1h, 8.4 min/ 1h,	3 min/0.25 h	feeding 5 times a week, resting 2.5 week	feeding 5 times a week, resting 2.5 week and 1 week	an hourly flush of 10 L of greywater.	21 s / 30 min
Q	L/h	5.56 L/h	4 L/30 min	7 L/min	135 L/d	5 L/d	2.5 and 5 L/d	10 L/h	15 L/h
A	m^2	substrate: 4 * 0.3 beds: 4 * 0.32	2_0.17	0.03	VF: 4x0.02, HF: 3x0.02	n/a	0.18	0.01 per pot	2.34
HLR	m/d	All beds: 0.11 First bed:0.44	n/a	0.1, 0.14, 0.19	135 Ln/ad	0.11, 0.22	0.055, 0.11	1.0 for first line, 0.096 for all infiltration area	0.67
HRT	h	18	n/a	45.6, 33.6, 24	5 days of 1 circulation per day	24, 48, 96	48, 96	n/a	n/a
BOD	%	>74	n/a	>80	1st day: VF:26, HF:1 2nd day (module 1,2): 51, 62	n/a	97	1_8-81 and 2_15-86	98
COD	%	>70	n/a	>90	n/a	n/a	n/a	1_7–80 and 2_14–86	>80

TN	%	n/a	pot: 91.8, block: 92.5	33-60	n/a	n/a	7-92	n/a	>30
TP	%	n/a	pot: 44.2, block: 40.4	n/a	n/a	n/a	7-85	n/a	>70
TOC	%	n/a	n/a	>80	n/a	n/a	73-89	n/a	n/a
TSS	%	n/a	pot: 98.4, block: 98.6	>90	n/a	n/a	88-95	n/a	>90
NH4 –N	%	>20	n/a	37.9-67	1st day: VF:61, HF:8 2nd day (module 1,2): 64, 65	n/a	n/a	19.4-70	n/a
NO3 –N	%	>72	n/a	no removal	1st day: VF:8, HF:10 2nd day (module 1,2): 15, 19	n/a	n/a	n/a	0.2-6.2
PO4	%	no removal	n/a	no removal	1st day: VF:33, HF:12 2nd day (module 1,2): 54, 47	n/a	n/a	n/a	n/a

Oxygen in green wall

Oxygen plays an important role in greywater treatment. The dissolved oxygen (DO) in the wastewater itself is considered negligible. Oxygen transfer into intermittent fed filter media systems is supplied from three sources: dissolved oxygen present in wastewater, convection due to intermittent dosing, and diffusion processes (Torrens *et al.* 2009a).

Avoiding clogging in green wall

The operation performance of green wall systems can be affected by a substantial decrease in hydraulic conductivity over time as a result of clogging (Le Coustumer *et al.,* 2009; Molle *et al.,* 2006; Hatt *et al.,* 2009). Clogging can happen due to the (1) decreased porosity of filter media (Blazejewski and Murat-Blazejewska 1997; Langergraber *et al.* 2003), the (2) rate of microbial growth and (3) suspended solids production. Therefore, in order to avoid overfeeding and clogging the system, some designing criteria need to be concerned, such as hydraulic loading rate (HLR), organic loading rate (OLR) and hydraulic retention time (HRT), all based on flow and treatment area of the filter. In any design, the wetland should follow a pre-treatment to prevent clogging from quickly causing problems (Fitch, 2014).

When applying larger doses on the upper portion of the filter, adsorption processes will be less effective due to higher velocities through the upper section, causing removal to decline. This implies that a greater depth cannot compensate for the loss of residence time through the first vital 10 cm filter depth where most of the pollutant oxidation occurs (Stevik *et al.*, 1999).

If the influent contaminated water is characterised with a higher organic load then the HLR may need to be decreased in order to avoid considerable reduction in infiltration capacity e.g., by having a larger system or by lowering the feeding dosage. In addition, a decrease in dose volume is accompanied by an increase in HRT and thus increased treatment performance (Boller *et al.* 1994; Schwager and Boller 1997; Bancolé *et al.* 2003; Stevik *et al.* 1999; Torrens *et al.* 2009b). In terms of the total HLR, the optimum value varies with filter media choice, strength of wastewater applied to the filter, method of dosing, etc., but recommended values that would be informative for greywater treating green walls are available. The USEPA (2002) reports the typical HLR for intermittent sand filters treating full strength domestic wastewater as 0.04-0.08 m/d. Norsk Rørsenter (2006) reported that a sand filter surface area slightly over 1m^2 with a depth of 75 cm would be sufficient for greywater treatment of a household of four persons when estimating rates of greywater production of around 100 L PE^{-1} d^{-1} (Jenssen 2002). However, these recommendations have to be reconsidered since such filter depth is not representative for green wall designs, especially if applied on green walls with horizontal flow, with smaller depths and larger upper areas.

Dosing of greywater in green wall

Repeated studies of green wall industry practitioners and literature recommend several irrigation periods during one day for more uniform delivery of water and nutrients to the plants. The reason behind this is that higher fractionation (i.e. smaller and more frequent application) of the total load of wastewater to the filter surface increases the removal efficiency of pathogen indicators (Prodanovic *et al.,* 2020, Torrens *et al.* 2009b; Ausland 1998; Stevik *et al.* 1999) and gives a greater reduction of COD and oxidation of nitrogen (Bancolé *et al.* 2003; Boller *et al.* 1994). Contrary, larger and less frequent doses can transport unoxidized material quickly through the depth of the filter. Therefore, enough time must pass between dosing to allow for effluent infiltration and redistribution, otherwise an almost completely saturated flow regime will develop (Schwager and Boller 1997). Additionally, it has been noted that very high fractionation of the wastewater load encourages biofilm development to concentrate at the very surface of the filter resulting in a higher risk for clogging, versus a lower fractioning of the wastewater load leading to more even biofilm development over the depth of the bed (Bancolé *et al.*

2003). Another way of clogging prevetion is by forcing the organisms into a starvation phase between dosing phases, in this way excessive biomass growth can be decreased and porosity increased (Tilley*et al.,* 2014). The USEPA recommends a dosing schedule of 12-24 times per day (USEPA 2002), correlated to differing grain size, as smaller grain sizes have higher moisture retention and require more time for the water to infiltrate before the next dose application.

7.3 Substrate

The main requirements regarding substrates, are related to (1) water retention, (2) light weightiness and (3) capacity to support plant growth. Studies of other vegetated filtration systems such as biofilters have demonstrated that media plays a critical role as it provides the physical support for plants and facilitates the primary removal processes for pollutants (Bratieres *et al.,* 2008). Media selection was found to be a much more dominant factor than plant selection for treatment performance (Pradhan *et al.*, 2019). For suspended solids and organics removal, any sand-based living wall system will provide excellent removal i.e. >80% for TSS and >90% for BOD (Fowdar, *et al.* 2017). Although biofilters have previously been designed to treat greywater, media such as sand and gravel used in these systems is different from media found in green walls (Fowdar *et al.,* 2017). Main feature of green walls is their vertical installation on the buildings wall. This design feature is hard to implement, due to weight increase caused by stored water in the saturated zone at the bottom of the filter, therefore green walls require lightweight media in order to reduce the load on their supporting structures (Oberndorfer *et al.,* 2007).

Lightweight materials such as perlite, vermiculite, coir, rock-wool, foam, and potting soil are commercially used for green wall construction. Water and air retention capacities and most physical properties of these media types are well understood (Papadopoulos *et al.,* 2008; Londra, 2010). However, when designing greywater treating green wall the focus should be on choosing a substrate with treatment abilities rather than its lightweightness. Results from Kadlec and Wallace (2009) showed that sand filtration systems are suitable for wastewater with turbidity below 50 NTU but are not lightweight. Pumice stone and kanuma soil could be potential candidates for green wall media because of their good performance in phosphorus (Karimaian *et al.,* 2013) and metal removal (Bhakta and Munekage, 2012). Other suitable media for greywater treatment are hydraulically slow coir, biochar rockwool and fytofoam, hydraulically fast perlite, vermiculite, growstone, expended clay and river sand (Prodanovic *et al.* 2017; Pradhan *et al.,* 2019). Prodanovic *et al.* 2017 found perlite to have the best hydraulic and treatment performance among the fast media while coco coir was found to be the best slow media. Prodanovic *et al.* (2018) tested different coir to perlite ratio mixes (1:3, 1:2, 1:1, 2:1, 3:1, 4:1) and showed that all of the hydraulically slower media mixes can successfully treat greywater. Perlite and coir have already been successfully trialled for removal of metals (Shukla *et al.,* 2009), dyes (Vijayakumar *et al.,* 2012), suspended solids by some previous studies (Todt *et al.,* 2014). However, caution should be given when choosing appropriate mixture in a particular water treatment system, as the plant selection and the type of irrigation system will also influence the suitable coir to perlite ratio (Prodanovic *et al.*, 2018). Also, coco coir degradation over time should be considered, since it might change the substrate porosity.

It is proposed to select one organic filter media to ensure organic carbon availability so that the denitrification process takes place if needed and one mineral filter media to potentiate the adsorption and precipitation of phosphorus (Fowdar *et al.*, 2017). In order to select a material with potential on releasing organic carbon, the C:N ratio should be considered. The recommended C:N ratios may vary depending on the type of system, type of wastewater and organic source. For example, Hang *et al.* (2016) recommended C:N ratios of at least 4:5 and 1.8:3.0 for TWs and bioreactor, respectively. Park *et al.*

(2008) results showed the maximum removal of nitrogen at 2:1 ratio. Therefore, additional carbon source is needed and can be provided by adding organic growth media such as coconut coir or fibres. Prodanovic *et al.* (2018) then used a 1:2 mix of perlite and coco coir as a substrate in all of his experimental designs .Biological processes were found to be the dominant mechanism for nitrogen and COD removal in coir, which provided sufficient retention time for denitrification processes. For perlite with lower retention times, physico-chemical processes dominated removal, showing the importance of media properties. Masi *et al.* (2016), tested LECA plus sand and LECA plus coconut fibres as porous media. Coconut fibres seemed to perform better than sand and this aspect can be related to the longer retention time provided by the adoption of this filling material in the pots. Pradhan *et al.* (2019), achieved removal percentages greater than 90% for all contaminants monitored (organics, solids, nitrogen and phosphorus) when using high surface area, small-diameter media such as coco coir, spent coffee grounds and sand. However, coco coir contains tannin, due to its organic nature and when saturated with water, it tends to leach colour. Nevertheless, the outflow colour levels observed in the experiment by Prodanovic *et al.* (2020) were still around the potable water value of 50 Pt/Co.

7.4 Vegetation

The treatment process of greywater not only depends upon the media material, but also the plants and microorganisms in the living wall. It is broadly agreed that microbial action is the dominant mechanism for most pollutant removal, and the impact of plants is minimal. Some treatment wetlands (submerged filters), particularly for metals, have no plants at all. Despite the minimal direct impacts of plants, planted wetlands do outperform nonplanted controls (Fitch, 2104).

Vegetation in a vertical green wall operates as a biofiltration system and provide treatment processes such as (1) oxidation, (2) filtration, (3) sedimentation, (4) adsorption, (5) microbial assimilation, and (6) microbial activity (Pradhan *et al.,* 2019). The physical presence of vegetation in filter media results in temperature buffering and additional surface area for attached microbial growth in the root zone (Stottmeister *et al.* 2003).

Wetland plants provide oxygen to the root, and some of this oxygen diffuses into the subsurface, which increases rates of organic degradation and nitrification by microbes at the root surface (Hansel *et al.,* 2001). In case of increased non-degradable coarse particles in the wastewater it is recommended to select plant species with effective underground thick rooted-systems with the ability to increase hydraulic performance, help break the clogging layer and maintain porosity through the creation of macropores (Le Coustumer *et al.,* 2012) which may be exacerbated during drying (Pham, 2015; Payne *et al.,* 2015; Molle *et al.,* 2006).

The growth of wetland plants will result in the uptake of N, P, and other elements as well as fixing carbon and transpiring water (Fitch, 2014). During the flowering, development of the fruit and growth period, generally late spring and early summer, as well there is significant uptake of nutrients and the photosynthesis is very intense to supply organic carbon to plants.

Concerning the carbon pollution, it may be that plants intake some carbon from their environment (Tagliavini et al, 2005). The research developed by Marchi *et al.* 2015 showed that CO_2 uptake by plant biomass is 0.44 – 3.18 kg CO_2 eqm^{-2} of vertical garden per year.

However, during senescence in the fall and early winter, perennial plants withdraw nutrients to the roots, and annuals concentrate nutrients into seeds. It should also be noted that nitrification and denitrification rates may decrease over time as plants nutrient uptake diminish once plants are past their active growth phase (Kadlec and Wallace, 2001). The detritus produced by plants is low in nutrients, so over the long term, plant growth results in little net nutrient uptake (Fitch, 2014). As senescent or dead plant mass decays, the residual complex polymeric material tends to enhance the organic content of the sediment. Such sediment accumulation is estimated to range from 0 to 2 cm/yr of new material at the top of the bed, with the rate of accumulation related to strength of the water treated (Fitch, 2014). To prevent cycling of nutrients, plants need to be harvested while still green (Fitch, 2014).

Some negative impacts of vegetation have been documented as well. An extensive root system inside the media filter can potentially clog the pore system (Stottmeister *et al.* 2003). Very high transpiration rates in warm climates can lead to a more concentrated effluent, especially in terms of TSS and salinity (Stottmeister *et al.* 2003; Coleman *et al.* 2001). Vegetated systems require specific maintenance routines including harvesting dead plant material since the breakdown of plant material can increase organic and nutrient loads to the effluent.

7.4.1 Plant selection

Nowadays a range of plants is used for vegetated walls, however there is a difference between plants selection for aesthetical purpose, or for wastewater treatment. It is important to select plants that tolerate water-logged conditions but also a high nutrient environment and elevated salinity (Fowdar *et al.*, 2014). Castellar Da Cunha *et al.* (2018) recommended to select plants that have good adaptation, provide ecosystem services and are socially accepted.

Green walls can support a large variety of plants, such as ferns, small shrubs and perennial flowers but ornamental species were usually utilized (Castellar Da Cunha *et al.,* 2018). A summary of different types of plants used in green wall and green roof systems and outcomes of different scientific studies can be seen in Table 15, collected by Pradhan *et al.* (2018) and a list of vegetation used in wastewater treatment green walls, can be found in Table 16, which was collected during the literature research for this master thesis.

Table 15: A summary of different types of plants used in green wall and green roof systems collected by Pradhan *et al.* (2018)

Location	Plant	System
Seville, Spain (Fernandez-Cañero *et al.*, 2013)	Sedum spp., succulent plants, ornamental grasses, herbs, turf lawn, shrubs, and small trees (*Prunus cerasifera Ehrh. and Cupressus sempervirens L.*)	Green roof
USA (Jungels *et al.*, 2013)	*Stoloniferous grasses, Sedum spp.,* and mixed perennial plant	Green roof
Venice, Italy (Mazzali *et al.*, 2013)	*Juniperus communis, Sedum spurium, Geranium sanguineum, Geranium Johnson's blue, Anemone sp., Viva minor, Parthenocissus tricuspidata, Heuchera micrantha Palace Purple, Salvia nemorosa, Lonicera pileata, Pittosporum tobira, Rosmarinus officinalis, Alchemilla mollis, Bergenia cordifolia, Oenothera missouriensis*, and *Plumbago capensis.*	Living wall
Spain (Azkorra *et al.*, 2015)	*Helichrysum thianschanicum*	Living wall
UK (Cameron *et al.*, 2014)	*Prunus laurocerasus, Prunus stachys* and *Hedera*	Living wall
Japan (Koyama *et al.*, 2013)	*Ipomoea tricolor, Canavalia gladiata, Pueraria lobata, Momordica charantia,* and *Apios americana*	Living wall
France (Musy *et al.*, 2017)	*Solene-microclimat*	Living wall, Green roof and lawns
Melbourne, Australia (Fowdar *et al.*, 2017)	*Carex appressa, Canna lilies, Lonicera japonica,* and ornamental grape vine	Living wall
Pune, India (Masi *et al.*, 2016)	*Abelia, Wedelia portulaca, Alternenthera, Duranta,* and *Hemigraphis*	Living wall

Table 16: Vegetation used in green walls wastewater treatments, by different studies

	Vegetation	
Green wall in Master's Thesis (2021)	*Plectranthus scutellarioides, Spathiphyllum wallisii, Chlorophytyum capense, Philodendron hederaceum, Euphorbia tithymaloides, Epipremnum aureum, Philodendron scandens Brasil*	
Prodanovic *et al.* (2020)	*Carex appressa, Nephrolepis obliterata* (Queen fern), *Liriope muscari* (Giant Lily turf), *Ophiopogon japonicus* (Mondograss)	
Zraunig *et al.* (2019)	*Helophytes, graminoids, tropical and subtropical plants: Cyperus alternifolius L., Monstera deliciosa, Carex acutiformis, Ficus pumila L., Juncus inflexus L., Philodendron scandens, Juncus effusus L., Philodendron erubescens, Equisetum hyemale L. Syngonium podophyllum, Spathiphyllum wallisii. Iris laevigata, Spathiphyllum wallisii "sensation", Mentha aquatica L, Calathea sp.*	
Castellar Da Cunha *et al.* (2018)	VF: *Salvia officinalis, Helycrisum italicum and Santolina chamaecyparissus*	HF: *Berula erecta, Iris pseudacorus and Juncus effusus*
Fowdar *et al.* (2018)	Native, ornamental, and climbing species. 1_Standard Saturated Zone: *Carex appressa, Canna lilies Phormium, Boston Ivy, non-vegetated*	2_Novel Saturated Zone: Grape vine, *Strelitzia reginae*
Fowdar *et al.* (2017)	Climbers: *Vitis vinifera (*Grape vine), *Parthenocissus tricuspidata* (Boston Ivy), *Pandorea jasminoides, Billardiera scandens*	Non-climbers: *trelitzia nicolai, Phormium spp. Canna lilies, Strelitzia reginae, Lonicera japonica, Carex appressa, Phragmites australis,*
Rysulova *et al.* (2017)	*Cotoneaster dammeri, Blechnum spicant, Carex oshimensis, Ophiopogonplaniscapus*	
Masi *et al.* (2016)	*Abelia, Wedelia Portulaca, Alternenthera, Duranta, Hemigraphis*	
Svete *et al.* (2012)	Lettuce, marigolds, spinach, cabbage	
Francis and Lorimer (2011)	*Adiantum raddianum, Chrysantheium morifolium dieffenbachia spp., Dracaena godseffiana, Epipremnum aureum, Hedera helix, Marraya sp., Nephrolepis exaltata, Philodendron sp., Rhododendron obtusum, Sansevieria trifasciata, Spathiphyllum maunahoa, and Vriesea splendens*	

Most of research on plant integration into substrate filter media systems such as TW have been conducted with species of marsh plants, especially reeds, *Phragmatis australis* being a typical wetland plant. These types of plants are extremely productive and their special adaptation to saturated conditions involves a transfer of oxygen into the root zone (Stottmeister *et al.* 2003) which can enhance organic matter, phosphorus, and nitrogen removal (Gikas and Tsihrintzis 2012; Torrens *et al.* 2009a). For example, a study by Matamoros *et al.* (2007), reported that the vegetated system removed pharmaceuticals and personal care products more efficiently than the sand filter, likely due to enhanced oxygenation of the filter bed. Henderson *et al.* (2007) investigated the effects of various shrub and groundcover species on the treatment of stormwater runoff in biofiltration mesocosms. While vegetation was reported to make little difference in the removal of organic matter, the nitrogen and phosphorus removal were significantly better in planted systems, which was attributed to higher microbial activity and population of microbes occurring in the rhizosphere. In the studies by Wang *et al.* (2014), Francis and Lorimer (2011) (Table 16) it was recognised that a variety of different species in TWs may increase the removal of TN in wastewater. Fowdar, *et al.* (2017) investigated various plants listed in Table 16. The plants he selected were based on their ability to tolerate water-logged conditions, elevated salinity and a high nutrient environment. All plant species were able to achieve high TN removal efficiencies (>80%), except for *Phormium, S. reginae* and *P. australis*. *Carex appressa* and *Canna lilies* were the best performers.

According to the fact that green wall designed in this master thesis was located indoors, tropical, subtropical, and waterlogged plants were searched for rather than just focusing on wetland plants. For this experiment plants were not estimated to make a big influence on overall treatment performance and would therefore not be thoroughly analysed through the process. The only goal was to find commonly found species in garden stores which are not difficult to maintain, like watery environment and do not grow higher than 50 cm. However, a further research on the types of indoor ornamental moisture loving plants that are known for larger nutrient intake is proposed.

8 GREEN WALL: EXPERIMENTAL DESIGN

In a period of five months from June to October 2019 a pilot greywater treating green wall with heat exchanger for greywater heat transfer was designed and implemented in the lobby of the Faculty of Civil and Geodetic Engineering, Hajdrihova ulica 28, Ljubljana. The space is open with about four m high ceiling, staircase and ventilated due to two entrances in the ground floor of the building (Figure 17). The green wall was fed with synthetic greywater and designed to treat greywater in horizontal flow through permeable substrate filled in four rectangular cascading beds. The design of the green wall in Ljubljana and the recipe for synthetic greywater was selected based on a similar experimental green wall setup at the BOKU University (Universität für Bodenkultur Wien) in Vienna (Pucher *et al.*, 2020) with the aim of future comparison of results. In addition to this, a heat exchanger was designed and set up with the help from faculty of Mechanical Engineering Ljubljana to point the direction of further studies of heat transfer from greywater to sanitary water. However, heat transfer evaluation was beyond the scope of this master thesis. The technological scheme of the green wall is presented in Figure 21 and described in the following sections.

Figure 17: Green wall pilot system at the end of the experimental period and the space of its location

8.1 Materials and methods

8.1.1 Hydraulic load of the pilot green wall

The volumes and dimensions of the pilot green wall were calculated based on estimated hydraulic load, HRT, literature research, treatment wetland design and recommendations from BOKU research group.

The inflow greywater to the green wall was 133L/d. Given the variance of HRT in different studies presented in section 7.2 (Table 14), the shortest HRT was selected to be 18 h as a design parameter. The actual HRT changes with time due to the change in root density.

If selected HRT is 18 h then the volume of water in the green wall $V_{treated\ water}$ (see section 4.2 and Table 5 for greywater use per PE) is:

$$V_{treated\ water} = Q_1 * HRT = 133L/d * 18h = 100\ \mathrm{L} \tag{10}$$

The hydraulic load Q_I per hour in the I. Phase is estimated as:

$$Q_1 = \frac{V_{treated\ water}}{HRT} = \frac{100\ L}{18\ h} = 5.56\ L/h = 133\ L/d \tag{11}$$

During the experiment, the operational mode of the system was transformed into a batch mode with intermittent flow (turning the water pump timer on and off every 15 min). Therefore, the flow was later (in the phase II.) changed to Q_2 = 11 L/h, with V_{tot} = 133 L/day staying the same.

$$Q_2 = Q_1 * \frac{t}{t_{on}} = 5.56\ L/h * \frac{60\ min}{30\ min} = 11\ L/h \tag{12}$$

The wall height in this study was 2.49 m tall, which allowed easy access to the beds with a ladder for maintenance and monitoring purposes. A taller structure would make access more difficult, and maintenance work on plants at great heights would also need to be considered. The frame of the pilot green wall was treatment of two stainless steel beams with handles, that supported four stainless steel cascaded rectangular beds, each with a volume of 84 L (dimensions of length x height x width: 160 x 26 x 20 cm) and a vertical distance of 25 cm between each bed.

Given the selected hydraulic time of HRT=18 h, and the initial amount of greywater of 100 L to be treated in that time, 25 L was distributed in one cascade bed. These 25 L were added to the substrate volume with selected porosity of about 48%. A mix of perlite (2–6 mm), gravel (2–6 mm) and a small volume of coconut fibres in a ratio 1:1:0.02 was selected as a substrate for this experiment. The substrate porosity was measured to be n = 55%, however the green wall was designed based on n = 48% porosity due to precaution of clogging. The design parameters are summarised in Figure 18 and in sections below.

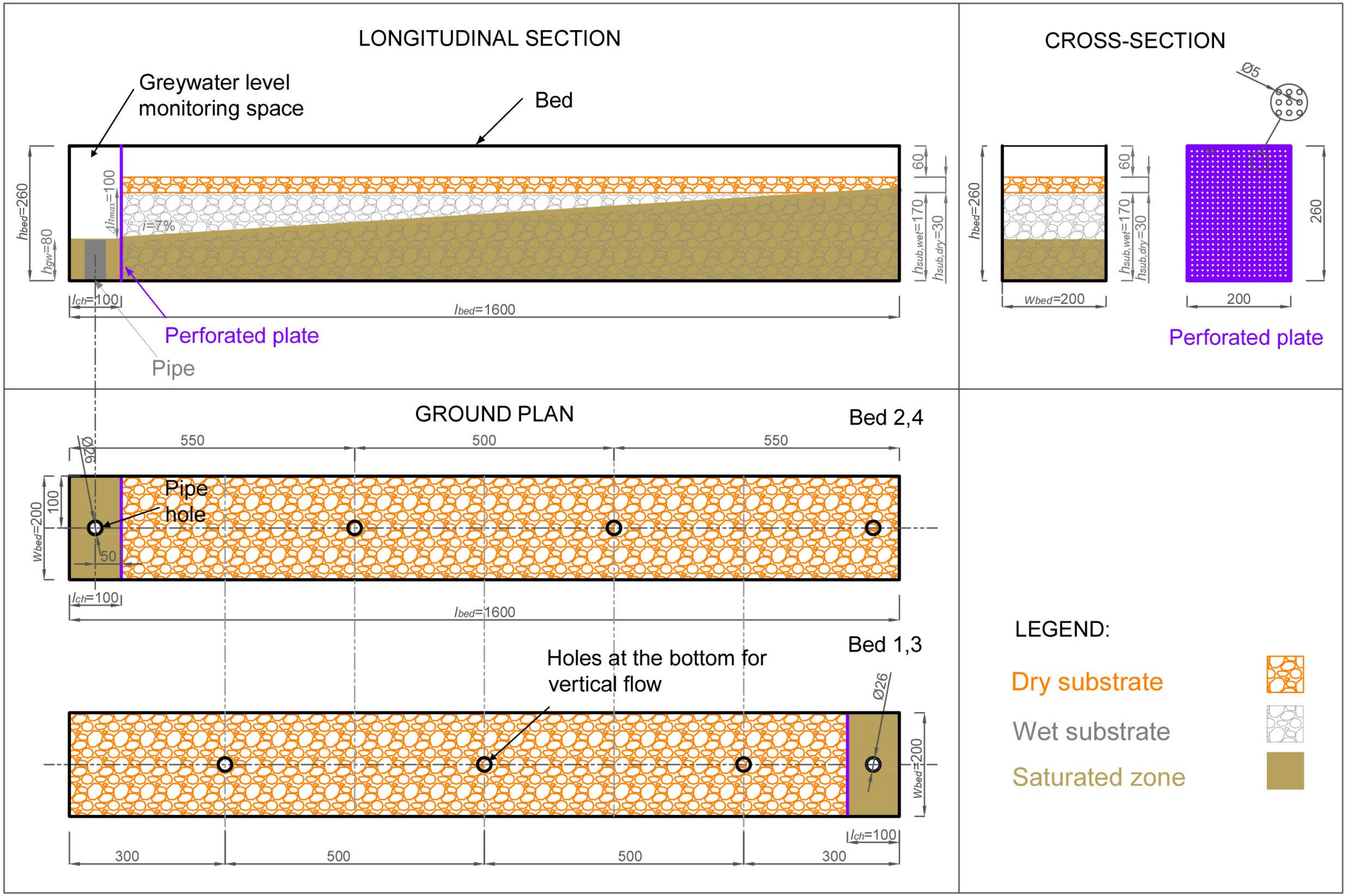


Figure 18: Dimensions of green walls bed, substrate, and demonstration of saturated zone in the substrate

8.1.2 Calculation of substrate dimensions

Firstly, the volume of wet substrate ($V_{sub,\ wet}$) in one bed was calculated from the known volume of water, which was approximately equal to the pore volume of the saturated zone $V_{gw} = V_n$=25 L and from the considered porosity in the design n = 48% (Table 17, Equation (13)).

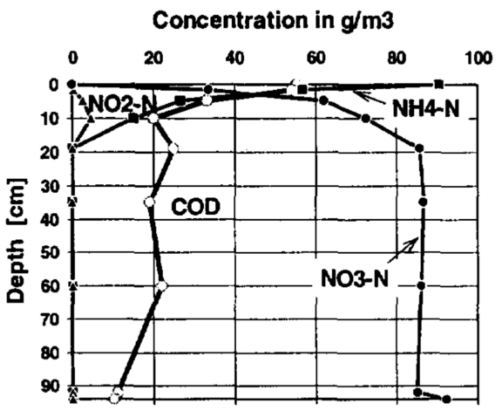
Table 17: Calculation of pore and substrate volume based on substrate porosity

Parameter	Bed (one)	Green wall (4 beds)	Unit	Description
n	48	48	%	…substrate porosity
$V_{gw} = V_n$	0.025	0.10	m^3	…volume of greywater or pore volume
$V_{sub,\ wet}$	0.052	0.21	m^3	…volume of wet substrate

Equation for calculation of substrate volume in one bed:

$$V_{sub,wet} = \frac{V_n * 100\%}{n} = \frac{0.025\ m^3 * 100\%}{48\%} = 0.052\ m^3 \quad (13)$$

Studies of intermittent filtration describe the nitrification process as happening at the very surface of filters, and of importance to a depth of 20 cm (see Graph 4) (Van Cuyk *et al.* 2001; von Felde and Kunst 1997; Widrig *et al.* 1996; Schwager and Boller 1997). Following that, the substrate was divided to bottom wet $h_{sub,wet}$ = 17 cm and upper additional $h_{sub,dry}$ = 3 cm dry substrate, due to the protection of plants from overwatering and to gain emptier pore space for higher oxygen intake, also needed for nitrification, in the upper layers of the substrate.



Graph 4: COD removal and nitrogen transformations with filter depth (Schwager and Boller, 1997)

The substrate length, height and width were calculated to be best 150 x 20 x 20 cm.

Table 18: Calculation of greywater level and substrate dimensions in the green wall's bed

SUBSTRATE DIMENSIONS				
Parameter	Bed (one)	Green wall (4 beds)	Unit	Description
h_{gw}	0.08	0.33	m	…maintained height of GW at the outlet of the bed
$h_{sub,\ wet}$	0.17	0.69	m	…height of wet substrate
$h_{sub,\ dry}$	0.03	0.12	m	…height of dry substrate
h_{sub}	0.20	0.80	m	…height of substrate, $h_{sub,\ wet} + h_{sub,\ dry}$
w_{sub}	0.20	0.80	m	…width of substrate
l_{sub}	1.50	6.00	m	…length of substrate
$A_{sub,\ vert.}$	0.04	0.16	m^2	…vertical area of the substrate
$A_{sub,\ surf.}$	0.30	1.20	m^2	…surface area of the substrate

8.1.3 Calculation of hydraulic conductivity (K_{sat})

The calculated minimum hydraulic conductivity in Table 19 describes the most extreme conditions for clogging estimated to occur. For its calculation I used Darcy's law, whose equation describes the flow of a fluid through a porous medium. This equation was also used to test whether the calculated dimensions of the beds and substrate (Table 18) were suitable.

Table 19: Calculation of hydraulic conductivity after clogging

Parameter	Quantity	Unit	Description
Δh_{max}	0.100	m	…max estimated head loss due to clogging by time
i	7	%	…hydraulic gradient (after clogging)
K_{sat}	$7.72*10^{-5}$	m/s	… hydraulic conductivity

$$i = \frac{\Delta h_{max}}{l_{sub}} = \frac{0.1\ m}{1.50\ m} = 7\% \tag{14}$$

$$K_{sat} = \frac{Q_{tot}}{A_{\text{sub,vert}} * i} = \frac{133\ L/d}{0.04\ m^2 * 7\%} = 5.68 * 10^{-4}\ m/s \tag{15}$$

When there is no clogging yet, the desired hydraulic gradient of the substrate was selected to be around $i = 1\%$.

Table 20: Calculation of hydraulic conductivity before clogging

Parameter	Quantity	Unit	Description
Δh	0.015	m	… natural estimated head loss of selected substrate
i	1	%	…hydraulic gradient (before clogging)
K_{sat}	$5.14*10^{-4}$	m/s	… hydraulic conductivity

$$i = \frac{\Delta h}{l_{sub}} = \frac{0.015\ m}{1.50\ m} = 1\% \tag{16}$$

$$K_{sat} = \frac{Q_{tot}}{A_{\text{sub,vert}} * i} = \frac{133\ L/d}{0.04\ m^2 * 1\%} = 3.79 * 10^{-3}\ m/s \tag{17}$$

Given the calculations above, pure sand with gravel, which has a good drainage and hydraulic conductivity above $K_{sat} = 3.79 * 10^{-3}$ m/s, was recognised as a suitable substrate for the design of this green wall (Table 21).

Table 21: Approximate value of hydraulic conductivity for different materials (Cedergren, 1977)

k (m/s)	10^{0}	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}	10^{-8}	10^{-9}	10^{-10}	10^{-11}
Drainage	good						bad		none			
Soil	pure gravel		pure sand (with gravel)			fine sand, silt, silted sand, clayed sand				clay		

The maximum head loss due to clogging with time was estimated to be Δh_{max} =10 cm. Using a Ø 26 mm plastic tube screwed to the bottom-outlet of the bed, the height of greywater was set to h_{gw} = 8 cm. This allowed to form an impoundment/saturated zone in the substrate. Basically, h_{gw} = 8 cm represents the height of 25 L of water per bed if there were no substrate and no flow present. Given that, to avoid spilling, the minimum hight of the beds should be at least those two values combined, that is 18 cm.

However, since the hight of the substrate was already set to $h_{sub} = h_{sub,\ wet} + h_{sub,\ dry} = 20$ cm, spilling was not expected.

8.1.4 Calculation of bed dimensions

The bed's length, height and width were calculated to be best 160 x 26 x 20 cm (Table 22). Before the exit of each bed a perforated steel sheet with round holes was put in to achieve a hand fitted l_{ch} = 10 cm length of water level check space without substrate, which stayed filled only with greywater. Its purpose was to easily observe the water level and to prevent clogging at the exit.

Table 22: Calculation of greywater bed dimensions

BED DIMENSIONS				
Parameter	**Bed (one)**	**Green wall (4 beds)**	**Unit**	**Description**
V_{bed}	0.084	0.337	m^3	…volume of bed
h_{bed}	0.26	1.054	m	…height of bed
w_{bed}	0.20	0.800	m	…width of bed
l_{bed}	1.60	6.400	m	…length of bed
$A_{bed,\ vert.}$	0.053	0.21	m^2	…side area of the bed
$A_{bed,\ surf.}$	0.422	1.69	m^2	…top area of the bed
l_{ch}	0.1	0.40	m	...length of the water level check space (hand fitted)
V_{ch}	0.0017	0.01	m^3	…volume of gw in water level check space

The materials in beds

Inside of the beds, a 2 mm DuPontTM Geoproma drainage geotextile fabric was placed first at the bottom and fitted at the sides of the bed to secure the substrate from washing away and to protect the plant roots from overheating in case of strong sun during the summer. Then a substrate was added over the geotextile to the bed height of 20 cm. To enable the denitrification process, I added coconut fibres as a source of carbon to the substrate. Two cm layer of coconut fibres was thoroughly cleaned and added on to the first 4 cm of the substrate only in the third bed. During the green wall's lifespan, the coconut is expected to disappear due to degradation, but slowly enough (high proportion of lignin-based compounds) to allow a full development of plant roots, which would guarantee proper green wall functioning.

Figure 19: Materials used in the third bed: 2 mm geotextile fabric, 4 cm of substrate, 2 cm of coconut fibres covered with 14 cm of substrate. First, second and fourth bed had no layer of coconut fibres.

8.1.5 Calculation of hydraulic loading rate (HLR)

Hydraulic loading rate per bed was calculated to be 0.44 m/d and per whole system of four beds 0.11 m/d (Table 23).

Table 23: HRT and HLR

Parameter	Bed (one)	Green wall (4 beds)	Unit	Description
HRT	18	18	h	…hydraulic retention time
HLR	0.44	0.11	m/d	…hydraulic loading rate

Hydraulic loading rate per bed:

$$HLR = \frac{Q_{tot}}{A_{bed,surf.}} = \frac{133\ L/d}{0.2\ m * 1.5\ m} = 0.44\ \mathrm{m/d} \tag{18}$$

Hydraulic loading rate per system (four beds):

$$HLR = \frac{Q_{tot}}{4 * A_{bed,surf.}} = \frac{133\ L/d}{4 * (0.2\ m * 1.5\ m)} = 0.11\ \mathrm{m/d} \tag{19}$$

8.1.6 Calculation of organic loading rate (OLR) and greywater preparation

A synthetic greywater mix was used in this experiment because of the large volumes required on a daily basis and to ensure consistency in composition and concentration for the duration of the experiment. Once a week a 900 L of greywater was prepared in a 1000 L tank (covered and protected from sunlight, potential algae growth and unwanted decomposition of greywater) manually by mixing sanitary water with shampoos and cleaning products by following the recipe from BOKU research group. Cleaning products used in a greywater recipe are presented in Table 24 and Figure 20.

Figure 20: Cleaning products used in a greywater recipe

Table 24: A list of cleaning products used in a greywater recipe

	Manufacturer	Product
1	Merck KGaA	Ammonium chloride (NH_4Cl)
2	Tandil	Multi power cleanser
3	Pril	Dishwashing liquid
4	Tandil	Dishwashing tabs
5	Tandil	Regeneration salt
6	Tandil	Detergent for clothes
7	Tandil	Softener

8	Ombia	Shampoo, shower gel
9	Ombia	Conditioner
10	Dentofit	Toothpaste
11	Ombia	Hand detergent

Calculated COD and BOD organic loading rates per first bed and per green wall are presented for all operational phases explained in section 8.2 (Table 25, Graph 13 and Graph 14). In the I. Phase, the OLR is calculated taking the input concentration in the mixing tank and dividing it by corresponding area of the bed, while in the II. and in the III. Phase the entering concentration was taken from the reservoir (i.e. lower concentration then in the mixing tank due to aeration.

Table 25: OLR calculated per one bed area and OLR calculated per all green bed area.

Phase	Parameter	First bed	Green wall (4 beds)	Unit	Description
I.	OLR_{COD}	272	68	g COD/m²d	…COD organic loading rate
II.	OLR_{COD}	118	29		
III.	OLR_{COD}	132	33		
I.	OLR_{BOD}	60	16	g BOD/m²d	…BOD organic loading rate
II.	OLR_{BOD}	18	5		
III.	OLR_{BOD}	27	7		

COD organic loading rate per bed:

$$OLR = \frac{[COD] * Q_{tot}}{A_{bed,surf.}} = \frac{[COD] * 133\ L/d}{0.2\ m * 1.5\ m} = see \text{ Table 25} \tag{20}$$

COD organic loading rate per system (four beds):

$$OLR = \frac{[COD] * Q_{tot}}{4 * A_{bed,surf.}} = \frac{[COD] * 133\ L/d}{4 * (0.2\ m * 1.5\ m)} = see \text{ Table 25} \tag{21}$$

BOD organic loading rate per bed:

$$OLR = \frac{[BOD] * Q_{tot}}{A_{bed,surf.}} = \frac{[BOD] * 133\ L/d}{0.2\ m * 1.5\ m} = see \text{ Table 25} \tag{22}$$

BOD organic loading rate per system (four beds):

$$OLR = \frac{[BOD] * Q_{tot}}{4 * A_{bed,surf.}} = \frac{[BOD] * 133\ L/d}{4 * (0.2\ m * 1.5\ m)} = see \text{ Table 25} \tag{23}$$

8.1.7 Technological scheme of the pilot green wall

After the greywater was prepared in the mixing tank, it was pumped into the 70 L heat exchanger and then to the 200 L reservoir from where it was pumped into the first (top) bed of the pilot green wall. The horizontal sub surface waterflow was then meandering through the substrate and was forced by gravity into the next cascading bed through the plastic tube, whereby a permanently high saturated water level was maintained, and exited into the last collecting tank (see Technological scheme in Figure 21). The system was not designed for greywater circulation, as it was important to have control over the input and output pollutant concentrations.

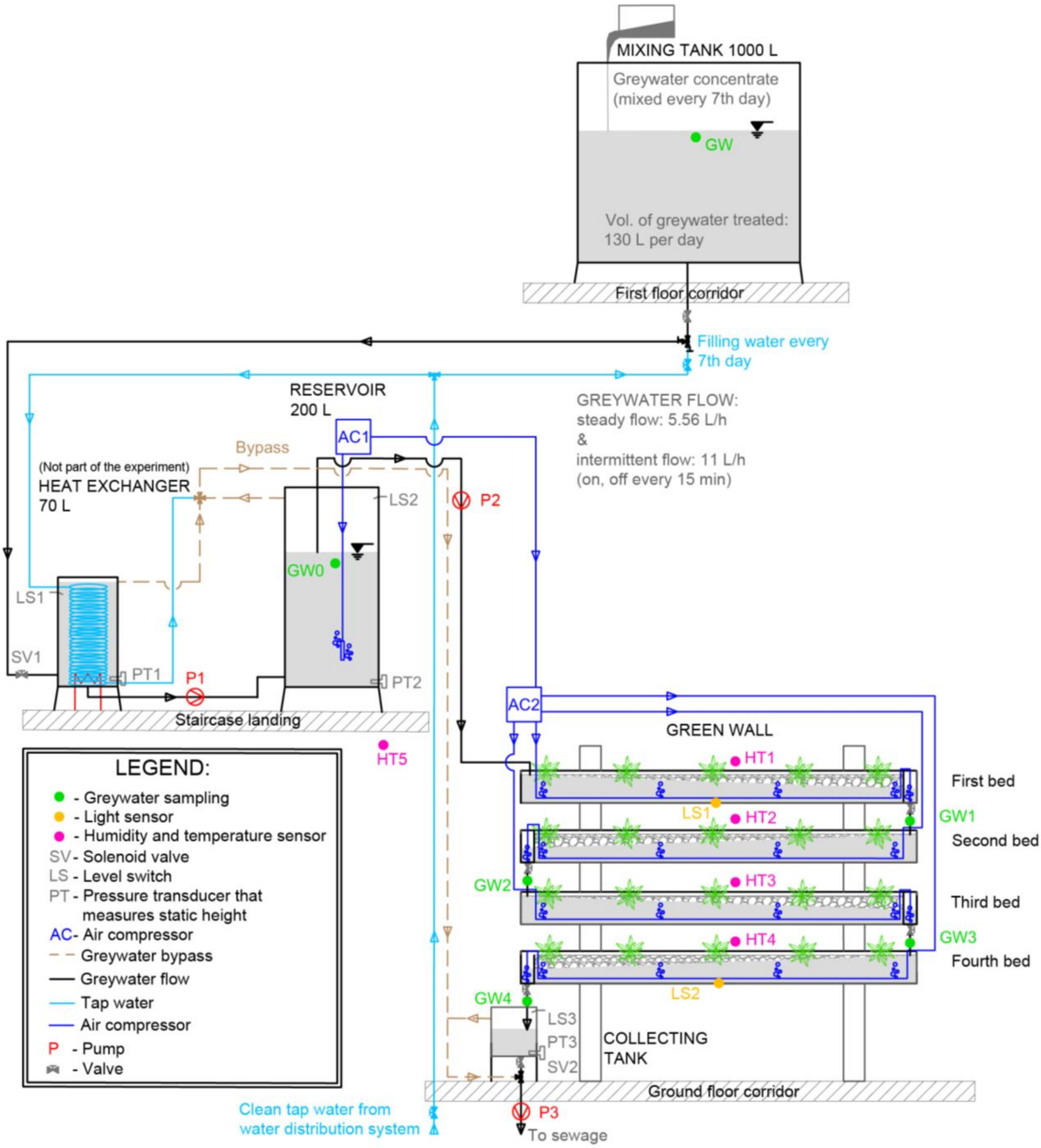


Figure 21: Technological scheme of green wall greywater treatment pilot system

8.1.8 Hardware and software

The system was run by hardware board and software Arduino which was programmed to maintain the water levels in the tanks. Also, the programme was used to start two external water pumps, open the solenoid valves, detect the leaking level in all the tanks with magnetic water level sensors and pressure transducers, measure the temperature, humidity and sunlight with sensors around the green wall and then log the data to computer. Apart from the equipment connected to the Arduino, a timer with peristaltic pump for setting the greywater flow and two air compressors (only in the II. and III. Phase) were used in the experiment. The location of specific equipment is seen in the Technological scheme in the Figure 21 and the list of equipment in Table 26.

Table 26: Equipment used in a system, its functions and technical specifications

Equipment	Function	Technical specifications
External pump 1	It pumps water from heat exchanger into the reservoir. It turns on and off according to the water level or depending on the level switch, pressure transducer and solenoid valve.	Voltage: 220–240 V Power: 650 W Q_{max}: 3800 L / h
Peristaltic pump	It pumps water from the reservoir into the first/top bed of the green wall. It turns on and off according to the timer (every 15 min during the intermittent flow).	Voltage: 90/260 V Power: 5 W Q: 1–12 L / h 50/60 Hz
External pump 2	It pumps water from the collecting tank into the drain. It turns on and off according to the water level or depending on the level switch, pressure transducer and solenoid valve.	220–240 V Power: 650 W Q_{max}: 3800 L / h
Air compressor 1	Blows air into the reservoir.	Power: 4.5 W
Air compressor 2	Blows air into the beds.	Power: 8 W
Solenoid valve 1, 2, 3	Starts the operation of external pumps. Opens/closes according to the set water level in the tanks. The opening and closing times of solenoid valves are set according to the water level detection of the level switch and the pressure transducer.	VXZ252HLA solenoid valve Withstand pressure: 1 MPa
Level switch 1, 2, 3	Serves as a safety catch in case the pressure transducer becomes inoperative. When the water level reaches the float, the float moves and an internal magnet activates a sealed reed relay in the device and actuates the external pump.	Contract Rating (Max): 50W Switching Voltage (Max): 220VDC Switching Current (Max): 1.5A Breakdown Voltage (Max): 300VDC Carry Current (Max): 3.0A Contract Resistance (Max): 100 OHM Temperature Rating: -20 - +80°C Material: PP
Pressure transducer 1, 2, 3	It is used to detect water levels in the reservoir, heat exchanger and sampling tank. It detects the pressure as electrical voltage created by the water height in the tank. It sends the analogue signal to a 10-bit A/D converter, which assigns pressure as a digital value of 0-1023. Therefore, additional calibration was required to determine the current mass of water in the tank.	Measuring range: 0.1–4 bar.
Humidity and temperature sensor 1, 2, 3, 4	It measures temperature and humidity behind every bed of the green wall. It sends an analogue signal via the I2C protocol to the Arduino, where the signal is calculated into digital data.	/
Humidity and temperature sensor 5	It measures temperature and humidity in the air 3 m away from the green wall. In this way the microclimate can be monitored and compared to the effects that green wall has on it.	
Light sensor 1,2	Measures intensity of the solar light oriented to the upper bed and the lowest bed of the green wall. Brightness is measured by measuring the current voltage. It acts as a “light resistor”. When there is more light, the electrical resistance decreases, when there is less light, the el. resistance increases. As a result, higher voltage induces less resistance and vice versa.	IR Sensor Spectrum: Wavelength: 550nm -1000nm (centered on 800) Visible Light Sensor Spectrum: Wavelength: 400nm-800nm (centered on 530) Voltage Supply: Power with 3-5VDC Output Type: I2C address 0x60 (7-bit) Operating Temperature: -40°C ~ 85°C

8.1.9 Sampling procedures

The experiment was divided into three phases, presented in Table 28, due to changing the concentration of greywater, implementing the air blowing into the greywater, changing steady flow into the intermittent flow and adding plants into the beds, over the experimental time. During each phase measurements of physical changes in the system and chemical analyses of organic matter and nutrient content, which showed the effectiveness of the wastewater treatment in the green wall, were performed twice a week or more when necessary.

The main measured physical parameters were: greywater flow entering the first bed, the temperature, oxygen, pH, redox and electrical conductivity of greywater in all the tanks and beds. Measurements of the physical parameters were carried out with Hach HQ40D Portable Meter and three additional probes seen in the Figure 22.

Figure 22: Hach HQ40D Portable Meter and three additional probes for measuring temperature, oxygen, pH, redox and electrical conductivity of the liquid

The main chemical parameters regularly observed were COD, BOD, NH_4-N, NO_3-N and PO_4-P. Also, NO_2-N and turbidity were measured irregularly just to get an insight. For analysing COD, NH_4-N, NO_3-N, NO_2-N and PO_4-P quick testers and spectrophotometer were used, whereas BOD was measured with a method that includes an electronic manometer, built into the lid of the respirometric bottle, that detects pressure changes (see Table 27, Figure 23, Figure 24, Figure 25, Figure 26).

Table 27: Sampling methods

Sampling method	Measured parameter
Hach Lange testers LCK 514	COD
Manometric respirometric method	BOD
Nessler method	Ammonia
Cadmium reduction method, using powder pillows	Nitrate
Diazotization method, using powder pillows	Nitrite
Amino acid method	Phosphorous
Hach HQ40D Portable Meter	Temperature, oxygen, pH, redox and electrical conductivity

Figure 23: Taken samples (left).

Figure 24: Hach Lange quick testers for measuring PO4-P

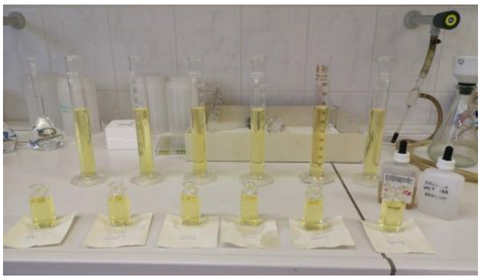


Figure 25: Hach Lange quick testers for measuring NH4-N

Figure 26: Measuring the BOD with a method that includes an electronic manometer (right)

8.2 Description of phases

Over a relatively short period of time, i.e. 127 days, from 30 October 2019 to 4 March 2020, a laboratory-pilot-scale experiment was conducted for the purpose of establishing suitable biological wastewater treatment conditions and to furthermore provide recommendations and determine the design parameters for greywater treating green wall systems. The whole experiment was performed in three phases depicted in Table 28.

Table 28: Description of phases

			I. (Start-up) Phase							**II. Phase**						**III. Phase**			
	Mo	Th	Fr	Mo	Tu	Tu	We	Th	We	Th	Su	Mo	Mo	Tu	Su	Mo	Th	Tu	We
Changes to the system	26.10.19	29.10.19	30.10.19	04.11.19	05.11.19	19.11.19	04.12.19	05.12.19	18.12.19	19.12.19	22.12.19	23.12.19	06.01.20	07.01.20	19.01.20	20.01.20	23.01.20	03.03.20	04.03.20
Experiment duration			127 days																
I. (start-up) phase			50 days																
II. Phase										32 days									
III. Phase																45 days			
Washing out the substrate (dilution)	4 days									4 days									
High GW organic and nutrient load			50 days																
Low GW (halved) organic and nutrient load												73 days							
Steady flow 5.56 L/h	73 days																		
Batch flow 11 L/h														58 days					
Dishwashing tabs: 6 →3 → 2			36 days					13 days		77 days									
Ammonium chloride added to GW						106 days													
Air blowing								91 days											
Planting and monitoring the plants																45 days			

I. Phase

After the system was build, a fresh water was added to the mixing tank without the cleaning products to flow through the system for four days to clear any dust and small particles out of the substrate. The first period of 50 days was a start-up phase. During the I. period, every seventh day the mixing tank was filled with fresh water accompanied with a mix of shampoos, cleaning products, softener, conditioner, ammonium chloride and salt as a concentrate, as listed in Table 29. However, the ammonium chloride was added later - three weeks in an experiment and the dishwashing tabs were halved from 5.5 to 3 tabs per mixture due to intense foaming. The initial input load of greywater was high in organic matter content, approximately 600 mg/L COD and 130 mg/L BOD.

Table 29: The type and amount of cleaning products used in a greywater concentrate in the I. Phase

			Concentrate		Greywater
	Manufacturer	**Product**	Mass [g]	Volume [mL]	Solution [mg/L]
1	Merck KGaA	Ammonium chloride (NH4CL)	26	26	0.029
2	Tandil	Multi power cleanser	95	95	0.105
3	Pril	Dishwashing liquid	119	118	0.132
4	Tandil	5.5, 3 x Dishwashing tabs (1x =21.63 g)	117, 65	115, 65	0.130, 0.072
5	Tandil	Regeneration salt	90	41	0.100
6	Tandil	Detergent for clothes	279	279	0.311
7	Tandil	Softener	132	126	0.146
8	Ombia	Shampoo, shower gel	497	477	0.553
9	Ombia	Conditioner	144	135	0.161
10	Dentofit	Toothpaste	27	23	0.030
11	Ombia	Hand detergent	103	103	0.114

A peristaltic pump was used to set the greywater steady flow to 5.56 L/h. No additional air blowers were added to the water at the start. Shortly after the launch (about two weeks later), the spread of unpleasant odours from the green wall and dropping oxygen concentrations in the greywater were detected. Odour issues have been previously reported for vegetated biofilters with submerged zones (Prodanovic *et al.,* 2020). Also, in the third bed greywater coloration was noticed possibly due to the coconut layer. However, coloration disappeared in the II. Phase (Figure 27).

Figure 27: Coloration in the third bed (left), no coloration in the fourth bed (right). Photos were taken on the same day 16. 12. 2020.

The O_2 concentrations were below 0.5 mg/L in all beds and NH_4-N was consistently higher at the outlet than at the inlet of the green wall. Concentrations of oxygen bellow 2 mg/L and higher concentrations of NH_4-N at the outlet indicated anoxic conditions, which were not intended at that extent (namely in all four beds) for this experiment, but they were expected due to high content of organic matter at the influent. To enable at least partial denitrification, the experiment was restructured so that only local anoxic zones exist. To prevent anoxic conditions in all four beds air blowing was introduced with 4.5 W and 8 W air compressors to the reservoir (2.25 W) and to the green wall (10.25 W) on 5.12.2019 - the 37. day of experiment. From that moment on the reservoir served as a buffer tank and as a pre-treatment tank, which allowed continuous load during the day and greywater homogenization. A small 4 mm air pipe was inserted in the middle of each bed along its entire length (see Figure 21). In this way the aeration was assured in the upper zone of the bed, however the lower part, close to the bottom may have turned into anoxic conditions. The airflow was controlled by observing the small bubbles from the pipe entering the monitoring space filled only with greywater at the end of the bed. However, despite the measures, the oxygen concentrations in the beds did not improve significantly due to too high concentrations of organics, which is not typical for greywater (particularly greywater excluding kitchen water - see Table 6, Table 7 and Table 8 in section Composition of greywater 4.4). Thus, the dosing of detergents and shampoos was adjusted so that more realistic concentrations of organic matter are achieved and consequently lower oxygen demand for greywater treatment. The substrate was then washed for four days with a flow of fresh water in order to reduce the organics and nutrient load on the green wall and to prepare the green wall for a new concentration of shampoos and cleaning products in the greywater, thus starting the II. Phase.

II. Phase

The II. Phase with halved greywater organic load and air blowing lasted for 32 days. The type and the new halved amounts of cleaning products used in a greywater concentrate in the II. Phase can be seen in Table 23. Foaming was detected already in the I. Phase and was still noticed in the II. Phase (see Figure 28). It was assumed that it was caused mainly by dishwasher tabs. Therefore, only 2 tabs were added into the greywater mix further on. The foaming in green wall stopped and minimised in the reservoir.

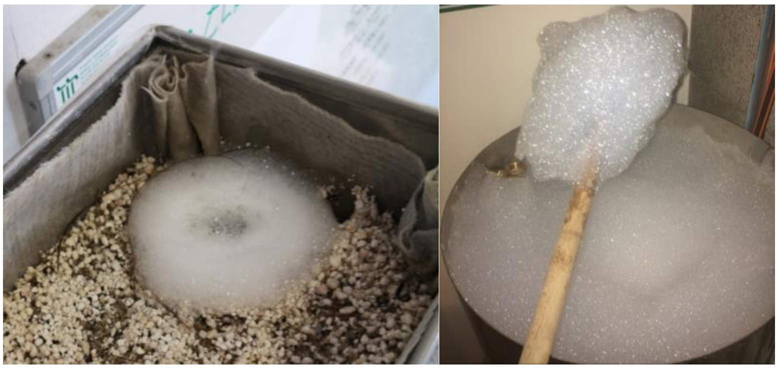


Figure 28: Foaming under the dripper (second bed, on the left photo). Intense foaming after aeration in the reservoir (right photo).

Table 30: The type and amount of cleaning products used in a greywater concentrate in II. and III. Phase

			Concentrate		Greywater
	Manufacturer	**Product**	Mass [g]	Volume [mL]	Solution [mg/L]
1	Merck KGaA	Ammonium chloride (NH4CL)	13	13	0.014
2	Tandil	Multi power cleanser	47	47	0.053
3	Pril	Dishwashing liquid	60	59	0.066
4	Tandil	2 x Dishwashing tabs	43	43	0.048
5	Tandil	Regeneration salt	45	20	0.050
6	Tandil	Detergent for clothes	140	140	0.155

7	Tandil	Softener	66	63	0.073
8	Ombia	Shampoo, shower gel	249	239	0.276
9	Ombia	Conditioner	72	68	0.080
10	Dentofit	Toothpaste	13	12	0.015
11	Ombia	Hand detergent	51	51	0.057

After the dilution, the oxygen concentrations in the greywater rose above 2 mg/L in almost all beds, which indicated the successful establishment of predominantly aerobic conditions in the wall. To ensure an oxygen concentration above 2 mg/L in all beds, the steady continuous flow regime of greywater through the wall was changed to the intermittent flow with a time-controlled water pump to load water from an oxygenated reservoir to the first bed for 15 min after every 15 min of rest. The flow was adjusted to the desired theoretical hydraulic retention time of 18 h and was therefore approximately 11.1 L/h. Transition to the batch system made it possible to raise and lower the water level in the substrate by creating a vacuum of air in the spaces between the pores. The oxygen concentration in the greywater in all beds rose above 2 mg/L and in the second and third bed occasionally above 4 mg/L. These measurements of higher oxygen concentrations ensured aerobic conditions throughout the green wall and minimized the perception of unpleasant odours.

III. Phase

The final or third period lasted for 45 days. During this period, data on nutrient removal was continuously collected, the plants were planted and their adaptation to the conditions in the wall observed. For the time of the experiment wilting and damage to the plants was almost not detected, moreover all 7 plants (*Philodendron hederaceum, Epipremnum aureum, Euphorbia tithymaloides, Philodendron scandens Brasil* - Sweetheart Plant, *Plectranthus scutellarioides, Spathiphyllum wallisii, Chlorophytyum capense*) produced shoots or offsprings. Therefore, suitable conditions for the growth of plants were achieved.

Figure 29: Plants and their conditions after the experiment. First row: *Plectranthus scutellarioides, Spathiphyllum wallisii* and *Chlorophytyum capense*. Second row: *Philodendron hederaceum, Euphorbia tithymaloides, Epipremnum aureum, Philodendron scandens Brasil*.

9 RESULTS AND DISCUSSION

In this chapter concentrations of COD, BOD, NH_4 –N, NO_3 –N, NO_2 –N, PO_4-P at selected control points along with oxygen (O_2), temperature, redox potential, HRT, HLR and OLR as well as their interdependencies are presented, discussed and analysed to assess the overall functioning of the green wall. Mean values, standard deviation, maximums, and minimums were calculated for each of the three phases to ease the evaluation and perception of the data, seen in the referenced graphs.

9.1 Chemical and biochemical oxygen demand (COD and BOD)

Summary of the results related to COD and BOD removal in all three phases are presented at six control points: mixing tank, reservoir, first bed, second bed, third bed and fourth bed in:

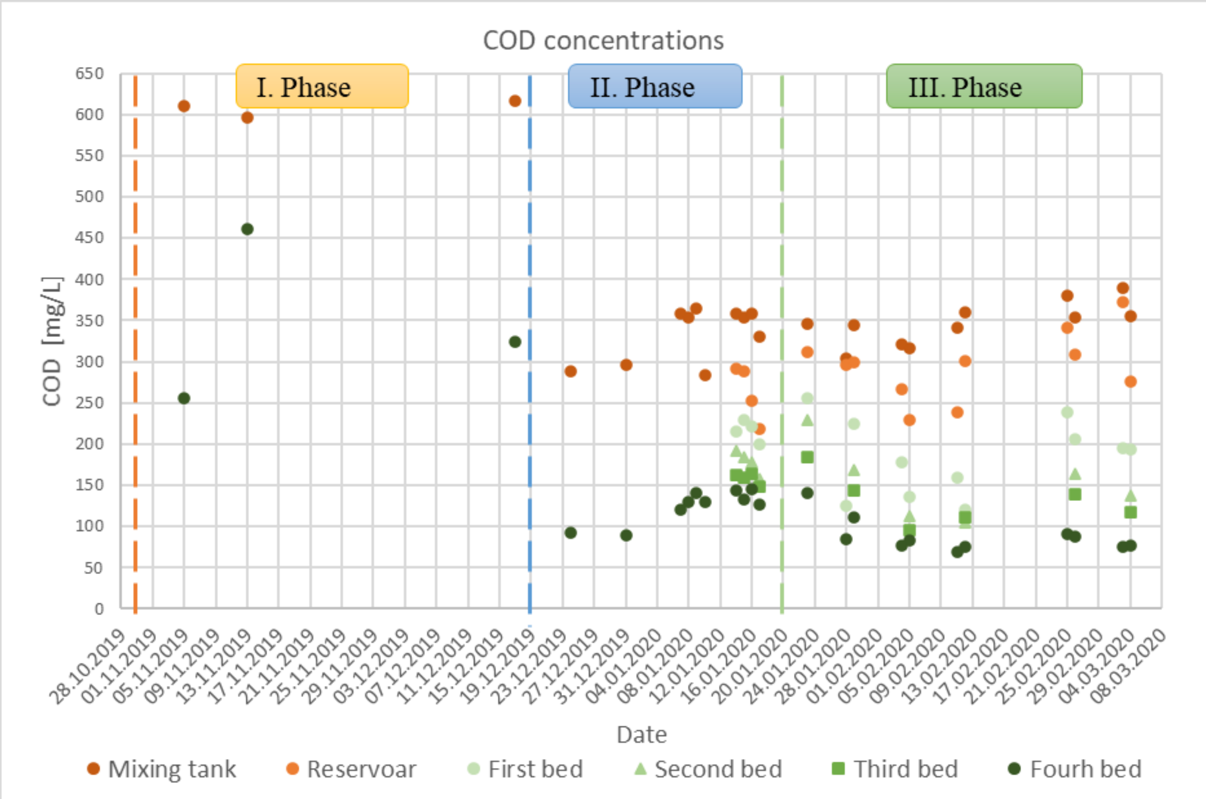
the graphs presenting COD values:

- Graph 5: Individual COD measurements in greywater in the mixing tank, reservoir, and the green wall beds according to the phases (I. Phase n = 6, II. Phase n = 36, III. Phase n = 56)
- Graph 7: The box plot of COD measurements in the mixing tank, reservoir, and the green wall beds according to the phases.
- Graph 9: The box plot of COD removal efficiency of greywater treatment between the mixing tank, reservoir, and the green wall beds according to the phases.

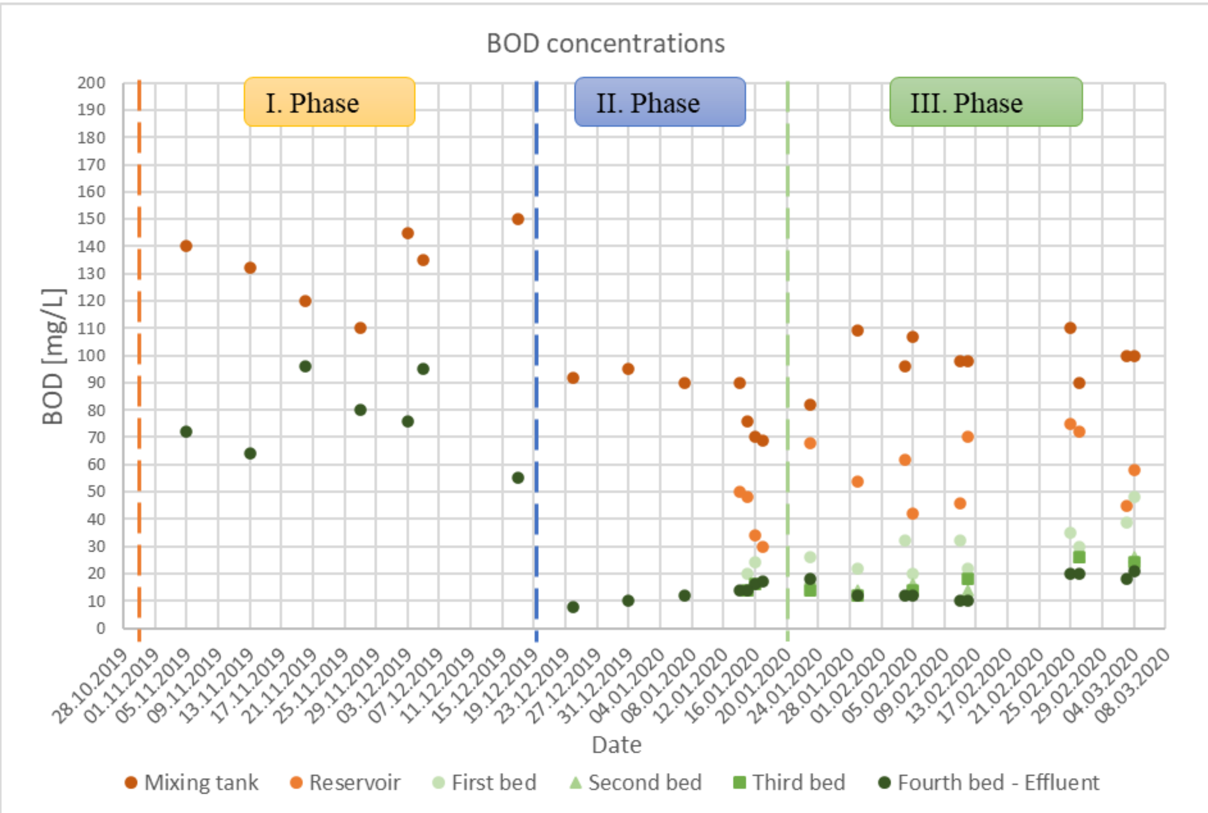
and in the graphs presenting BOD values:

- Graph 6: Individual BOD measurements in greywater
- Graph 8: The box plot of BOD measurements in the mixing tank, reservoir, and the green wall beds according to the phases.
- Graph 10: The box plot of BOD removal efficiency of greywater treatment between the mixing tank, reservoir, and the green wall beds according to the phases.

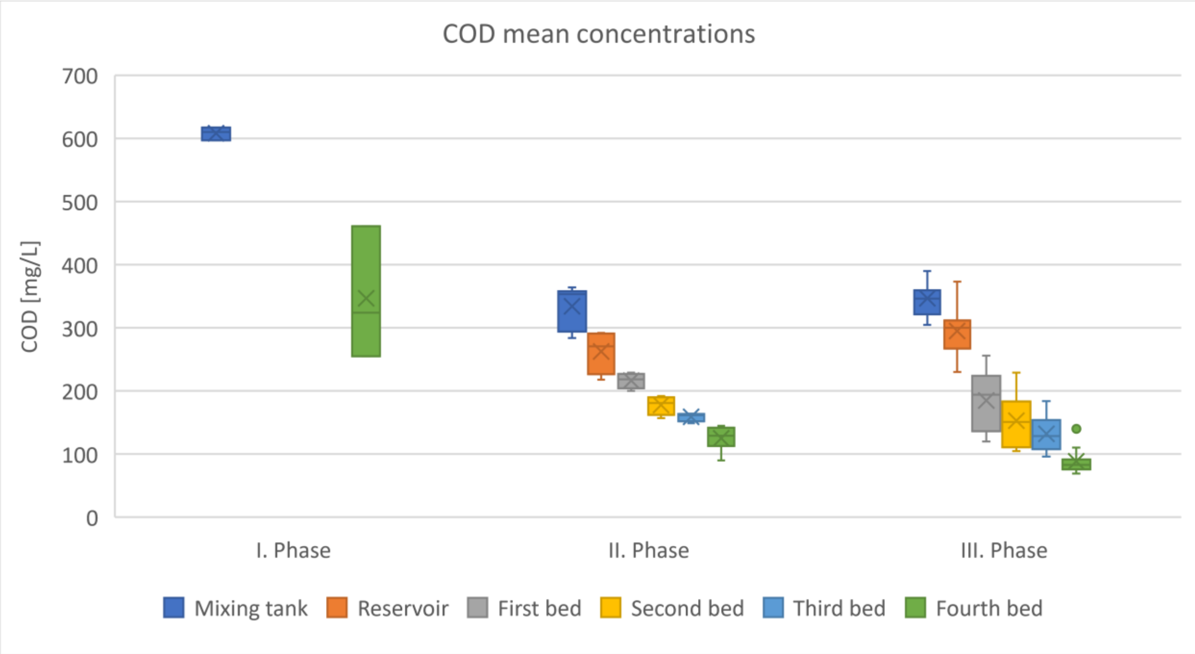
According to the oxygen concentrations, the three phases were divided by estimated established conditions in the substrate: I. anoxic and II., III. anoxic and aerobic (II., III. “anoxic/aerobic” written from here on).



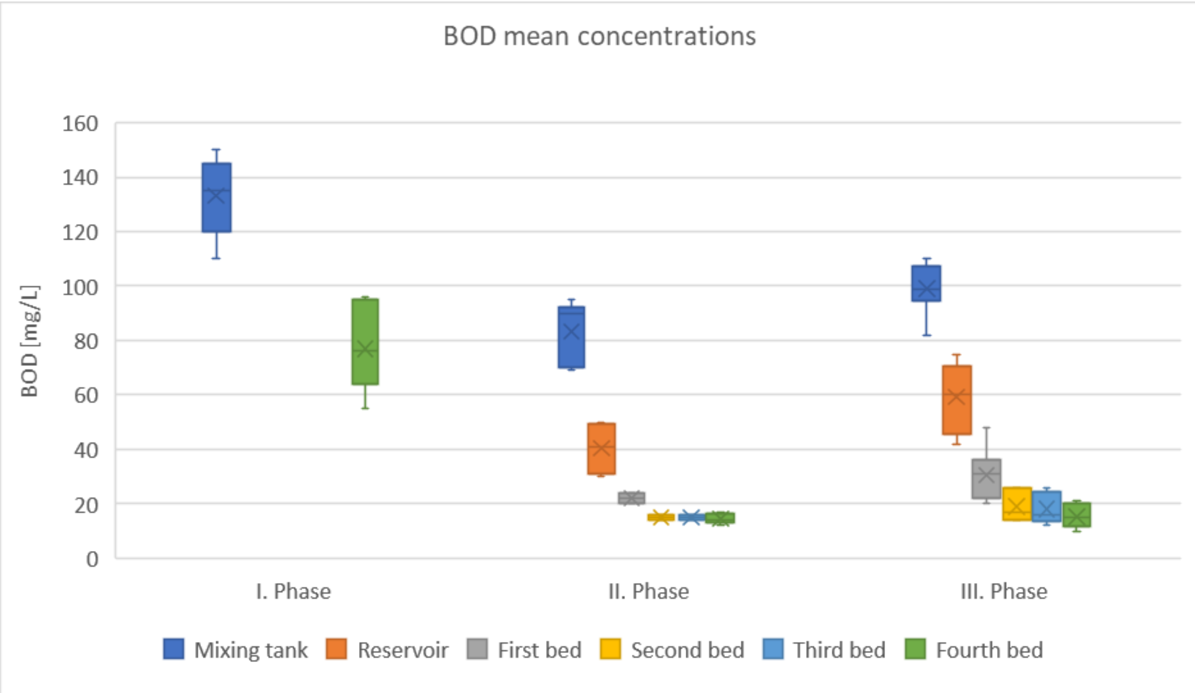
Graph 5: Individual COD measurements in greywater in the mixing tank, reservoir, and the green wall beds according to the phases (I. Phase n = 6, II. Phase n = 36, III. Phase n = 56)



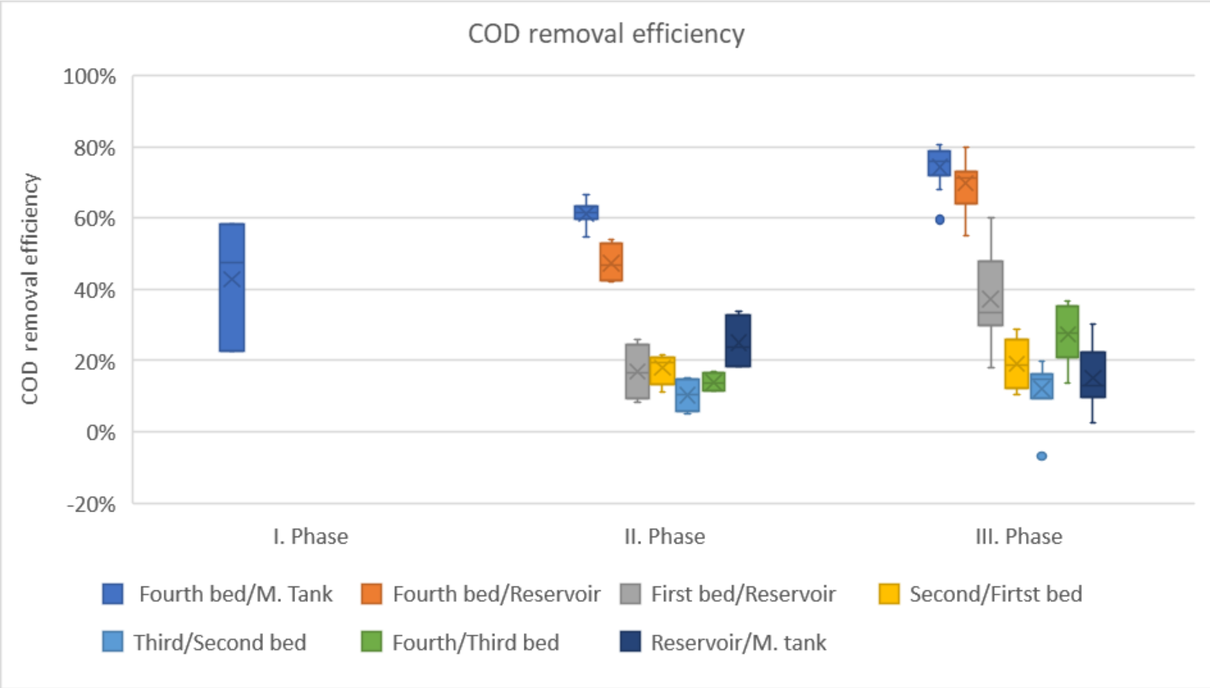
Graph 6: Individual BOD measurements in greywater in the mixing tank, reservoir, and the green wall beds according to the phases (I. Phase n = 14, II. Phase n = 24, III. Phase n = 52)



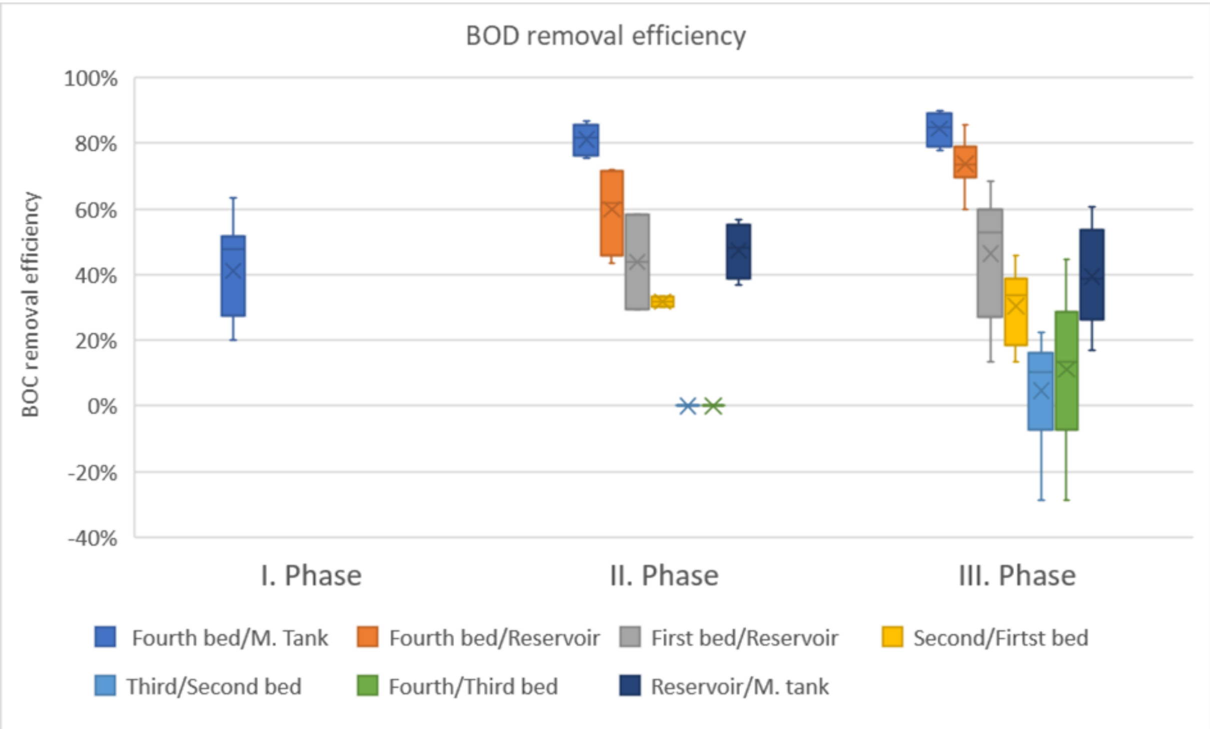
Graph 7: The box plot of COD measurements in the mixing tank, reservoir, and the green wall beds according to the phases.



Graph 8: The box plot of BOD measurements in the mixing tank, reservoir, and the green wall beds according to the phases.



Graph 9: The box plot of COD removal efficiency of greywater treatment between the mixing tank, reservoir, and the green wall beds according to the phases.



Graph 10: The box plot of BOD removal efficiency of greywater treatment between the mixing tank, reservoir, and the green wall beds according to the phases.

I. anoxic phase included measurements only from the mixing tank and the last bed of the green wall due to estimation that almost no degradation of greywater is taking place in the mixing tank. COD mean values in the mixing tank was in average 608 mg/L and after the treatment in the green wall it dropped to an average 347 mg/L. Whereas the initial BOD concentration was in average 133 mg/L and after the treatment in the green wall it dropped to an average 77 mg/L (Graph 7 and Graph 8). That is 261 mg/L of COD (43% efficiency) and 56 mg/L of BOD (41% efficiency) removal established in the I. Phase (see Graph 9).

In the II. anoxic/aerobic phase with halve of the greywater concentrations the average COD influent concentration from the mixing tank was 332 mg/L, from the aerated reservoir was then 263 mg/L and the average effluent value from the fourth bed was 134 mg/L. Also, the BOD load in the mixing tank in average was 83 mg/L, in reservoir 41 mg/L and after the treatment in the green wall it dropped to an average of 15 mg/L (Graph 7 and Graph 8). That was in average 87 mg/L COD removed from mixing tank to reservoir (25% efficiency), 126 mg/L of COD was removed from reservoir to green wall (47% efficiency) and 212 mg/L of COD was removed from mixing tank to green wall total (61% efficiency). Note also that, in average 36 mg/L BOD was removed from mixing tank to reservoir (47% efficiency), 25 mg/L of BOD was removed from reservoir to green wall (60% efficiency), 64 mg/L of BOD was removed from mixing tank to green wall total (81% efficiency) in the II. Phase (see Graph 9).

Lastly, in the III. anoxic/aerobic phase the average influent value from the mixing tank was 347 mg/L, from the aerated reservoir was 295 mg/L and the average effluent value from the fourth bed was 88 mg/L. Whereas, the BOD load in the mixing tank in average was 99 mg/L, in reservoir 59 mg/L and it stayed equivalent to the former phase in average of 15 mg/L at effluent (Graph 7 and Graph 8). That was in average 52 mg/L COD removed from mixing tank to reservoir (15% efficiency), 206 mg/L of COD was removed from reservoir to green wall (70% efficiency) and 258 mg/L of COD was removed from mixing tank to green wall total (74% efficiency). Next, 40 mg/L BOD was removed from mixing tank to reservoir (39% efficiency), 44 mg/L of BOD was removed from reservoir to green wall (74% efficiency), 84 mg/L of BOD was removed from mixing tank to green wall total (84% efficiency) in the III. Phase (see Graph 9 and Graph 10).

COD efficiency of fourth bed/mixing tank gradually increased from 43% in the I. Phase to 61% in the II. Phase and to 74% in the last phase. Also, BOD efficiency of fourth bed/mixing tank ratio increased from 41% to 81% and then to 84% according to three phases (see Graph 9 and Graph 10). To mention again, there were no measurements taken from the reservoir in the first (start-up) phase, since a relatively small difference in the organic load was assumed between the mixing tank and reservoir.

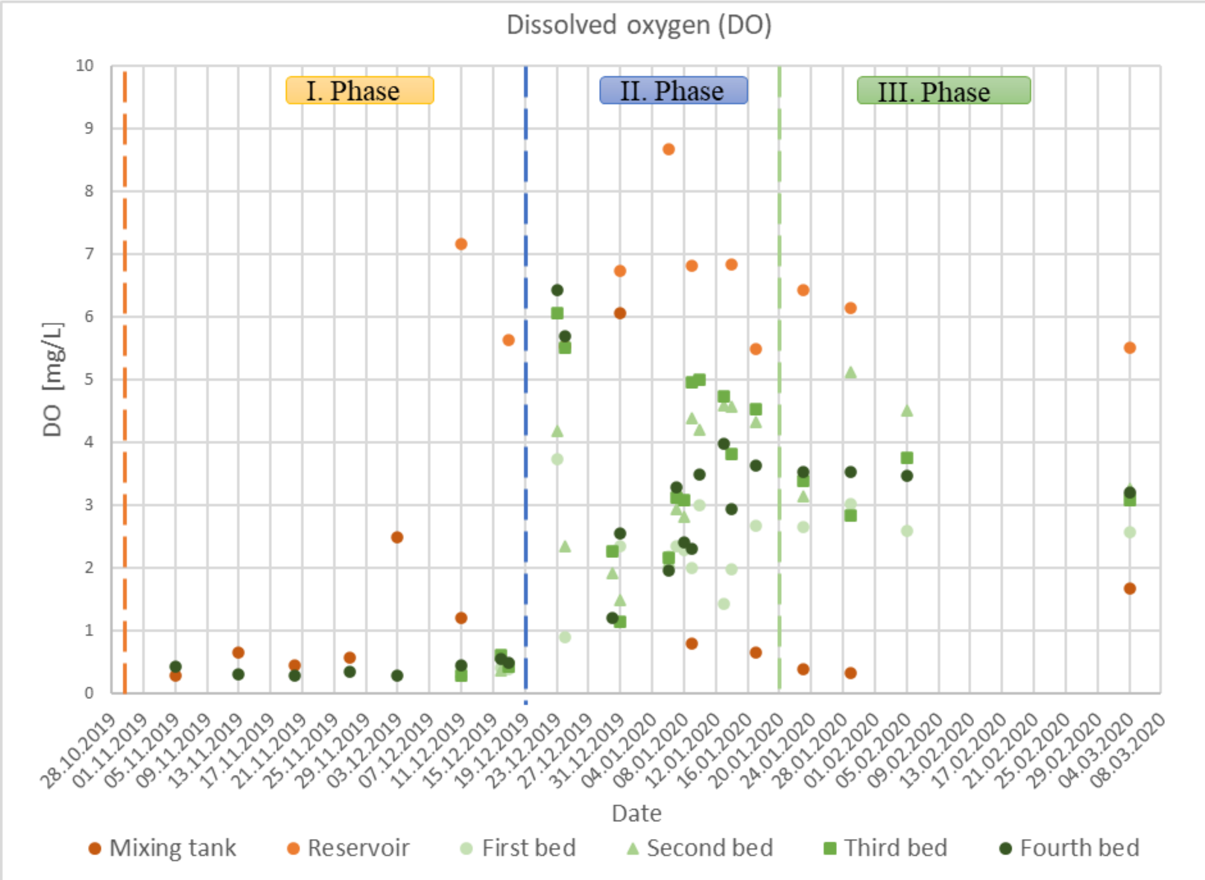
Reasons for low removal in I. Phase are: (1) the wall was operating under anoxic conditions (slower process rates) and (2) low BOD/COD ratio. The BOD:COD ratio ranged 0.26 to 0.3 and indicated that the influent greywater was less biodegradable than typical domestic wastewater. This means that the microorganisms need longer time for acclimatization, but in our experiment, there were only 4 months available for the acclimation in comparison to other studies on green wall with an experiment period of 8–22 months (Table 14). The BOD:COD ratio of treated effluent was 0.16 which indicates a good BOD removal.

Higher removal in II. and III. Phase was observed due to (1) aerobic conditions (intermittent flow and aeration) and (2) better acclimatization of microbes. It can be concluded that the green wall itself was capable to treat in average up to 206 mg/L of COD, 80% of the total COD and up to 44 mg/L of BOD, 54% of the total BOD in the system. Treatment ability of the reservoir from the II. Phase on became important as well due to aeration, since it performed 20–41% and 46–60% of overall COD and BOD treatment. However, this could be reduced by lowering the amount of air blowing, since the concentrations of oxygen in the reservoir were maintained high >6 mg/L.

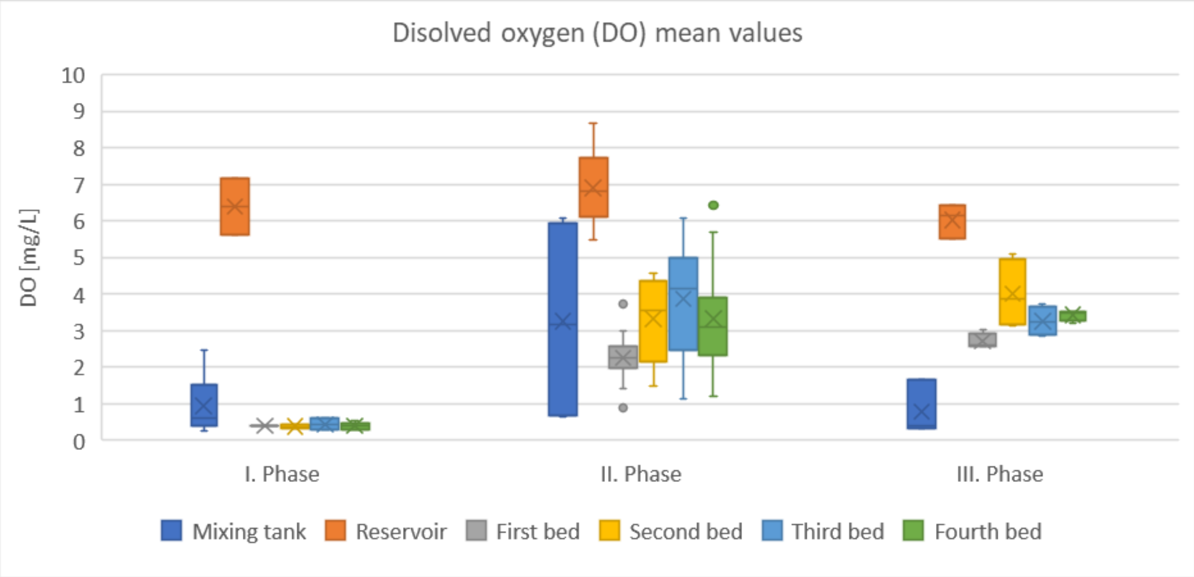
9.2 Dissolved oxygen (DO)

During the I. Phase of the experiment odour from the green wall was detected and was linked with anoxic conditions. Therefore, oxygen levels were measured in all three phases at six control points: mixing tank, reservoir, first bed, second bed, third bed and fourth bed, and are presented in:

- Graph 11: Individual DO measurements in greywater in mixing tank, reservoir, and in the green wall beds according to the phases (I. Phase n = 25, II. phase n = 57, III. Phase n = 22).
- Graph 12: The box plot of DO measurements in the mixing tank, reservoir, and the green wall beds according to the phases.



Graph 11: Individual DO measurements in greywater in mixing tank, reservoir, and in the green wall beds according to the phases (I. Phase n = 25, II. phase n = 57, III. Phase n = 22).



Graph 12: The box plot of DO measurements in the mixing tank, reservoir, and the green wall beds according to the phases.

In the I. Phase, the average level of oxygen in the mixing tank and in the reservoir was below 1 mg/L and in the green wall's beds was below 0.5 mg/L which confirmed that anoxic conditions were established (Graph 11, Graph 12).

After the application of air blowers in the II. Phase in average oxygen increased above 6 mg/L in the reservoir, above 2 mg/L in the first bed, 3 mg/L in the second and above 3 mg/L third and fourth bed (Graph 11, Graph 12).

The average oxygen concentrations stayed similar in the III. Phase with about 6 mg/L in the reservoir, 2.7 mg/L in the first bed, 4 mg/L in the second and above 3 mg/L in the fourth and third bed (Graph 11, Graph 12).

The oxygen in green wall was measured at the exit of the beds in a space only filled with greywater. Therefore, it was possible that oxygen levels in the substrate dropped near 0 mg/L and greywater treatment was happening under anoxic conditions and associated odours in the I. Phase. In order to further avoid anoxic conditions and associated odours, air blowers. Another thing that helped to increase oxygen levels in the II. Phase in the green wall beds, was changing the greywater flow from continuously steady to intermittent flow which influenced water to change levels in the substrate and by every water level drop the oxygen was transported into the substrate by suction. However, it can hardly be asserted that aerobic conditions were achieved everywhere in the substrate volume, since the redox potential stayed negative in all the beds even after applying the air blowers (Graph 31, Graph 32). On the other hand, aerobic conditions were most likely achieved in the first 5–10 cm of the substrate and then at the exit of the bed the upper greywater was mixed with the saturated anoxic greywater at the bottom of the beds and the analysis of redox potential was measured to be in anoxic state, below 0 mV respectively (Graph 31, Graph 32). An investigation of the dissolved oxygen gradient and redox potential inside the filter sections is recommended in future attempts to support this theory.

During the experiment, changes to the primary system design had to be made due to the reason of designing an indoor green wall. Namely, the initial concentrations of COD and BOD were very high, in fact quite higher than those found in typical greywater. This pushed the system to anoxic conditions, which resulted in unpleasant odour. For this reason, COD and BOD were accordingly adjusted and effluent values at the start of the II. Phase might appear lower due to the dilution (Graph 5, Graph 6).

These values were not included in the calculation of statistics and greywater treatment efficiencies which are presented in Graph 9 and Graph 10.

Looking back, it might be preferably to turn on the air blowers in the reservoir intermittently or reduce their operation, since they had a very big treatment effect towards lowering the BOD and COD values. However, this was not done during the experiment due to attempting to maintain the level of oxygen above 2 mg/L in all the beds to avoid odours, especially in the first bed where the oxygen concentrations were the lowest (Graph 11, Graph 12). The first bed appeared to achieve the most greywater treatment linked with the highest oxygen depletion in comparison to other beds which in some cases appeared to be an effective polishing step (Graph 9, Graph 10). To illustrate, the efficiency of the first bed was 17% for COD and 44% for BOD in the II. Phase and increased to 37% for COD and 46% for BOD in the III. Phase (Graph 9, Graph 10). To explain, aerobic and facultative microorganisms need oxygen for decomposition of organic matter and as described with the Monod kinetics, the more organically loaded water is, the faster microorganisms grow and the more oxygen they need. Therefore, the bed that received the most organically loaded greywater was by all means the first bed 272, 118, 132 g COD/m^{2}d and 60, 18, 27 g BOD/m^{2}d in I., II., III. Phase (Graph 13, Graph 14).

Since the first bed treated greywater so effectively, it could be stated that the last two beds could possibly be removed according to the II. and III. Phase measurements of BOD, where the treatment efficiency in the third bed was only 0% (II. Phase), 5% (III. Phase) and of the fourth bed was only 0% (II. Phase), 11% (III. Phase) (Graph 10). However, they proved their role in removal of slowly biodegradable organic matter. Due to long retention time the biomass could acclimatize and thus the COD removal in the last two beds were almost equally important as in the first two beds, where treatment in the third bed was 10% (II. Phase) and 12% (III. Phase) and of fourth bed 14% (II. Phase) and 37% (III. Phase) respectively. To compare, the efficiency of the first bed was 17%, 37% and of the second bed 18% and 19% in the II. and the III. Phase respectively (Graph 9).

Note also, that additional organic matter was probably released from the coconut fibres, introduced in the third bed. Most likely this impacted the low removal efficiency for BOD in the third and fourth bed. The leaching of organics and nutrients from coconut fibres was tested separately to assess the possible load effect on greywater treatment in the green wall. Ten grams of coconut fibres had been marinated in 1000 mL of deionised water overnight. The next day, measurements of NH_4 –N, NO_3 –N, NO_2 –N, PO_4-P, COD, BOD were taken and results are listed in Table 31. It can be stated that coconut fibres have some effect to the overall organics and nutrient leaching in greywater and have to be taken into account when analysing the removal efficiencies. In fact, on some measuring days BOD and COD values appeared to be higher in the third rather than in the second bed.

Overall, the organic matter released in the third and fourth bed from coconut fibres impacted more the BOD values than the COD, thus the removal efficiency was lower for BOD then the COD.

Table 31: Analysis of 10 g of coconut fibres soaked in 1000 mL of deionised water overnight

Coconut	Days of soaking	**NH_4 –N**	**NO_3 –N**	**NO_2 –N**	**PO_4-P**	**COD**	**BOD**
		mg/L	mg/L	mg/L	mg/L	mg/L	mg/L
	1	0.79	0.7	0.011	1.36	34.3	8

Furthermore, the water retention time in the mixing tank was fairly long and allowed for degradation of readily biodegradable organic matted (mostly BOD). In this experiment the greywater was prepared on Mondays - once per seven days. Therefore, the COD and BOD values from the first measuring days - Tuesdays were normally higher than on Wednesdays, thus the measurements later in the week were avoided. To prove this right, a series of COD and BOD measurements from Tuesday to Friday were taken. In four days, COD in the mixing tank dropped for 8% and BOD for 23% (see Table 32).

Table 32: Decreasing COD and BOD concentrations four days in a row due to the passing time

Day	Date	COD [mg/L]	BOD [mg/L]
Tue	14/01/20	358	90
Wed	15/01/20	353	76
Thu	16/01/20	358	70
Fri	17/01/20	330	69

After treatment, the effluent from the green wall design satisfied the existing guidelines for BOD urban reuse of most countries that already have them. Under US's standards for example which require <10 mg BOD/L for unrestricted urban reuse (non-potable applications with uncontrolled public access) and certainly environmental reuse requiring <30 mg/L in terms of BOD requirement and for certain reuses under Italy's standards in terms of COD requirement requiring <100 mg/L (see Table 4).

9.3 Organic loading rate (OLR) and clogging

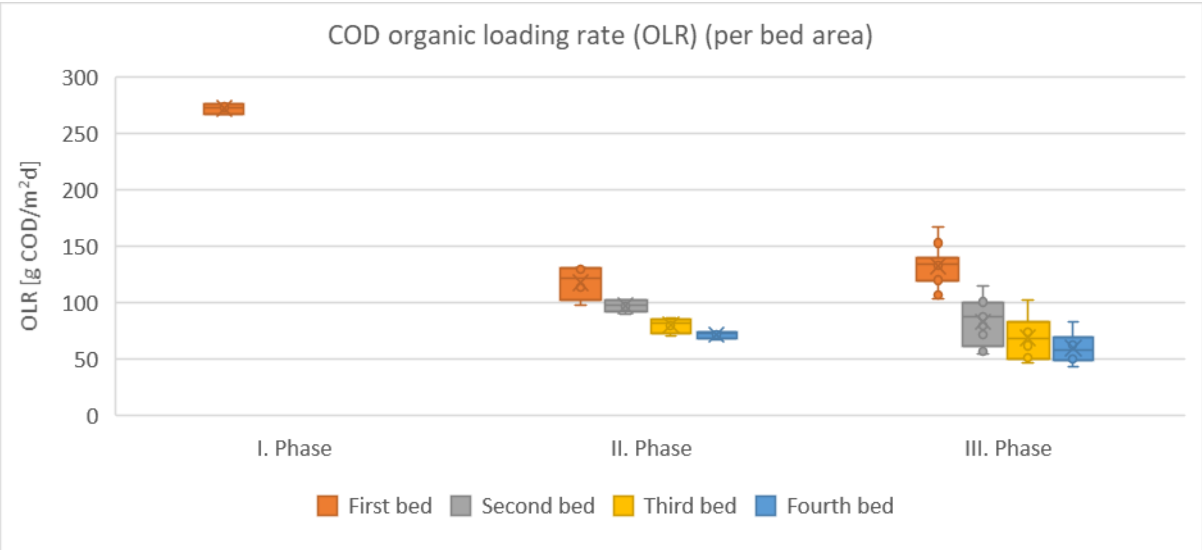
As described in the section 8.1.6 the organic load was calculated based on COD and BOD concentrations and the beds surface area. Summary of the results related to OLR in all three phases are presented at four control points: first bed, second bed, third bed and fourth bed and for the whole system in:

the graphs presenting OLR values:

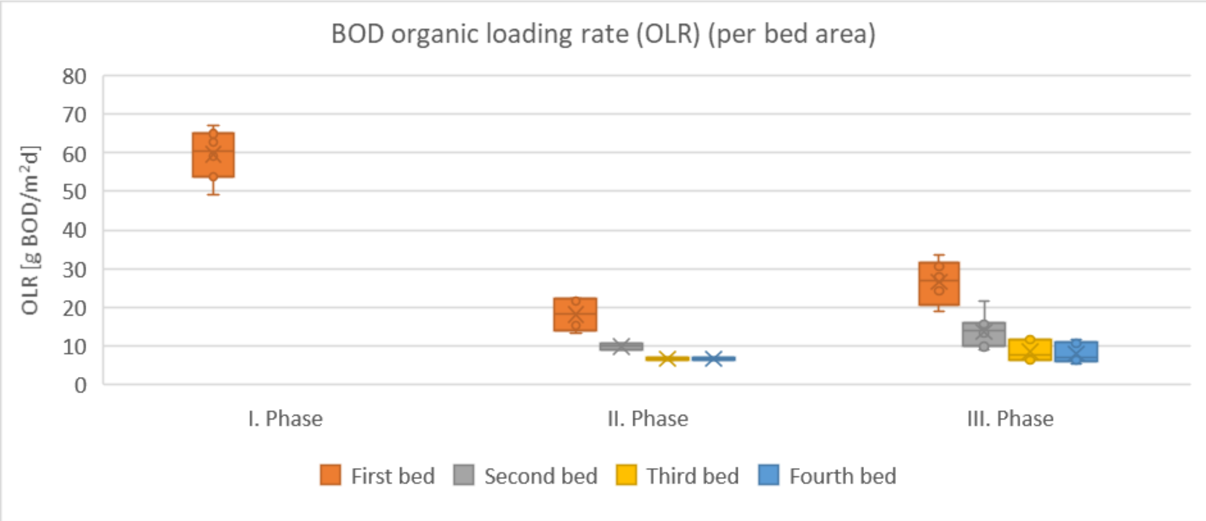
- Graph 13: The box plot of organic lading rate (COD) in the four beds. Inflow COD from the mixing tank was taken in the I. Phase and from the reservoir in the II. and III. Phase.
- Graph 14: The box plot of organic lading rate (BOD) in the four beds. Inflow BOD from the mixing tank was taken in the I. Phase and from the reservoir in the II. and III. Phase.

and the table presenting OLR values in the section 8.1.6:

- Table 25: OLR calculated per one bed area and OLR calculated per all green bed area.



Graph 13: The box plot of organic lading rate (COD) in the four beds. Inflow COD from the mixing tank was taken in the I. Phase and from the reservoir in the II. and III. Phase.



Graph 14: The box plot of organic lading rate (BOD) in the four beds. Inflow BOD from the mixing tank was taken in the I. Phase and from the reservoir in the II. and III. Phase.

In the I. Phase OLR_{COD} and OLR_{BOD} per one bed area were calculated for the first bed, i.e. 272 g COD/(m^2d) and 60 g BOD/(m^2d) (Graph 13, Graph 14).

In the II. Phase OLR_{COD} per one bed area was calculated for the first bed, second bed, third bed, fourth bed. The calculated OLR_{COD} values in the same order were: 118, 97, 80 and 71 g COD/m^2d. Whereas, the calculated OLR_{BOD} values in the same order were: 35, 18, 10, 7 and 7 g BOD/m^2d (Graph 13, Graph 14).

In the III. Phase OLR_{COD} per one bed area was calculated for the first bed, second bed, third bed, fourth bed and theoretically for the fifth bed. The calculated OLR_{COD} values in the same order were: 132, 83, 68 and 59 g COD/m^2d. Whereas, the calculated OLR_{BOD} values in the same order were: 27, 14, 9 and 8 g BOD/m^2d (Graph 13, Graph 14).

Hydraulic conductivity reduces over time as a result of clogging, due to the decreased porosity of filter media and the rate of microbial growth and sludge production.

Organic loading rate of previous studies were found to be 15.9–34 g COD/m^2d and up to 167 g COD/m^2d, whereas OLR in this study was set to be similar, in an average range of 68, 29, 33 g COD/m^2d and 16, 5, 7 g BOD/m^2d in I., II., III. Phase for all beds area combined (see Table 25).

Clogging was observed only at the top (first) bed which received the most organic load and therefore was prone to grow the largest biofilter (Figure 30). To illustrate, first bed received 272, 118, 132 g COD/m^2d and 60, 18, 27 g BOD/m^2d in the I., II., III. Phase and removed the most COD and BOD as well (Graph 9, Graph 10).

Figure 30: Clogging in the first bed

To compare, hydraulic conductivity, as an indicator for clogging, at the end of experiment was calculated for each of the beds. It was found that the first bed had the lowest hydraulic conductivity 4.6 x 10^{-4} m/s in comparison to other beds, 5.8 x 10^{-4} m/s was measured in the third and 6.4 x 10^{-4} m/s in the second, fourth bed. To further avoid that, it is recommended to lower fractionation of the wastewater load from 15 min of feeding (flooding) and 15 min of resting to for example, 15 min of feeding and 1 h of resting, since it has been noted that very high fractionation of the wastewater load encourages biofilm development to concentrate at the very surface of the filter resulting in a higher risk for clogging, versus a lower fractioning of the wastewater load leading to more even biofilm development over the depth of the bed (Bancolé *et al.* 2003). Applying the slope of 1% would also affect the faster relocation of greywater from the start of the bed to the exit due to gravity force.

9.4 Ammonium, Nitrite and Nitrate

Summary of the results related to NH_4-N and NO_3-N removal in all three phases are presented at six control points: mixing tank, reservoir, first bed, second bed, third bed and fourth bed in:

the graphs presenting NH_4-N values:

- Graph 15: Individual Ammonium measurements in greywater in mixing tank, reservoir, and in the green wall beds (I. Phase n = 14, II. Phase n = 32, III. Phase n = 56).
- Graph 17: The box plot of Ammonium measurements in the mixing tank, reservoir, and the green wall beds according to the phases.
- Graph 19: The box plot of Ammonium removal efficiency of greywater treatment between mixing tank, reservoir, and the green wall beds according to the phases.

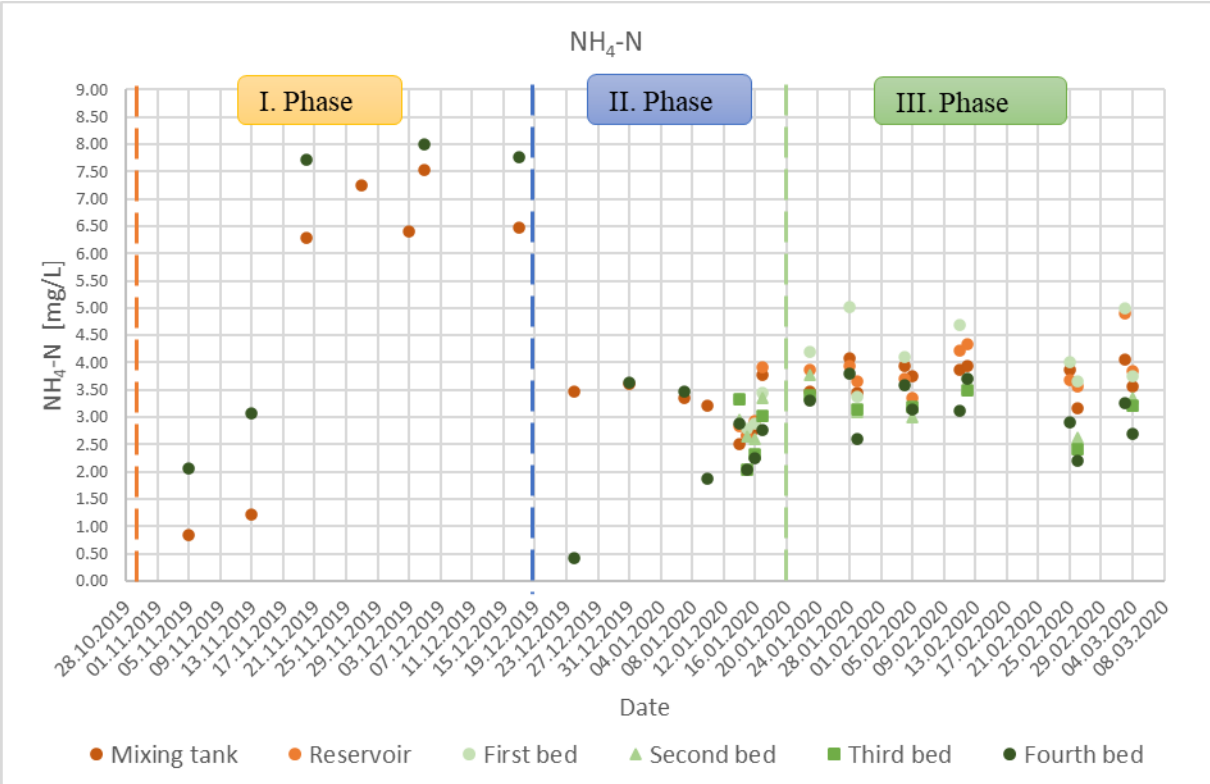
the graphs presenting NO_3-N values:

- Graph 16: Individual Nitrate measurements in greywater in mixing tank, reservoir, and in the green wall beds (I. Phase n = 14, II.phase n = 29, III. Phase n = 43).
- Graph 20: The box plot of Nitrate removal efficiency of greywater treatment between mixing tank, reservoir, and the green wall beds according to the phases.

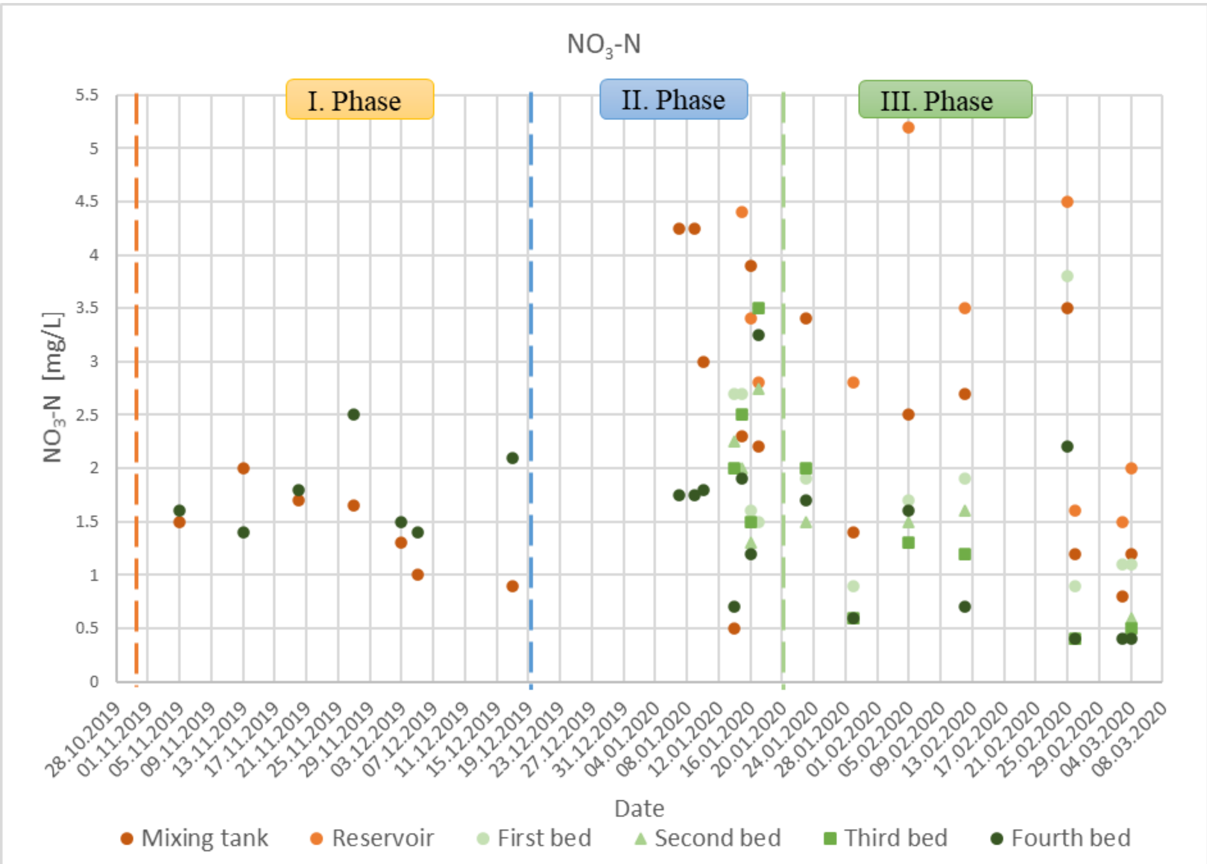
However, due to the negligible and unstable concentrations of NO_2-N values measured in the greywater in the I. Phase, the measurements later on and the statistical analysis were abandoned.

the graphs presenting NO_2-N values in the I. Phase:

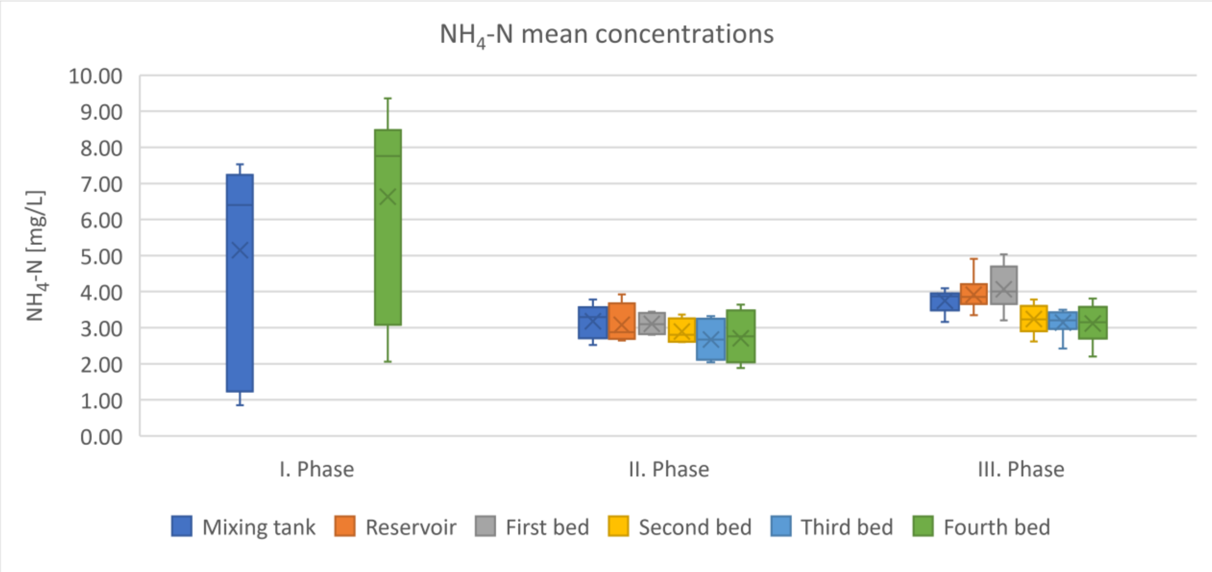
- Graph 21: Individual Nitrite measurements in greywater in mixing tank, reservoir, and in the green wall beds according to the I. Phase (I. Phase n = 14).



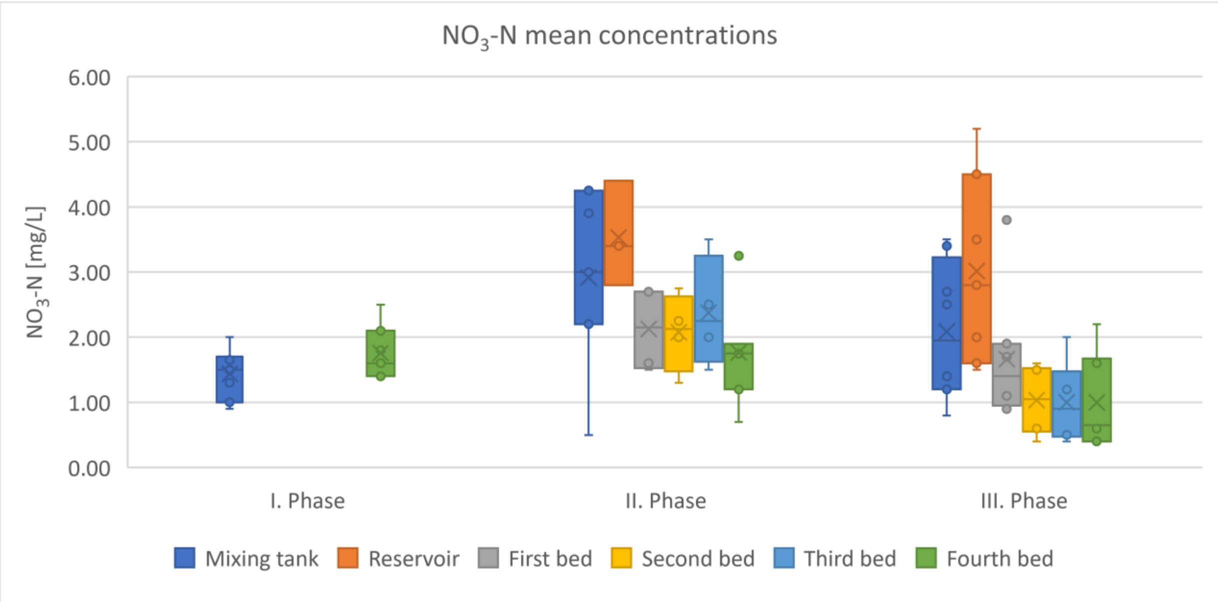
Graph 15: Individual Ammonium measurements in greywater in mixing tank, reservoir, and in the green wall beds (I. Phase n = 14, II. Phase n = 32, III. Phase n = 56).



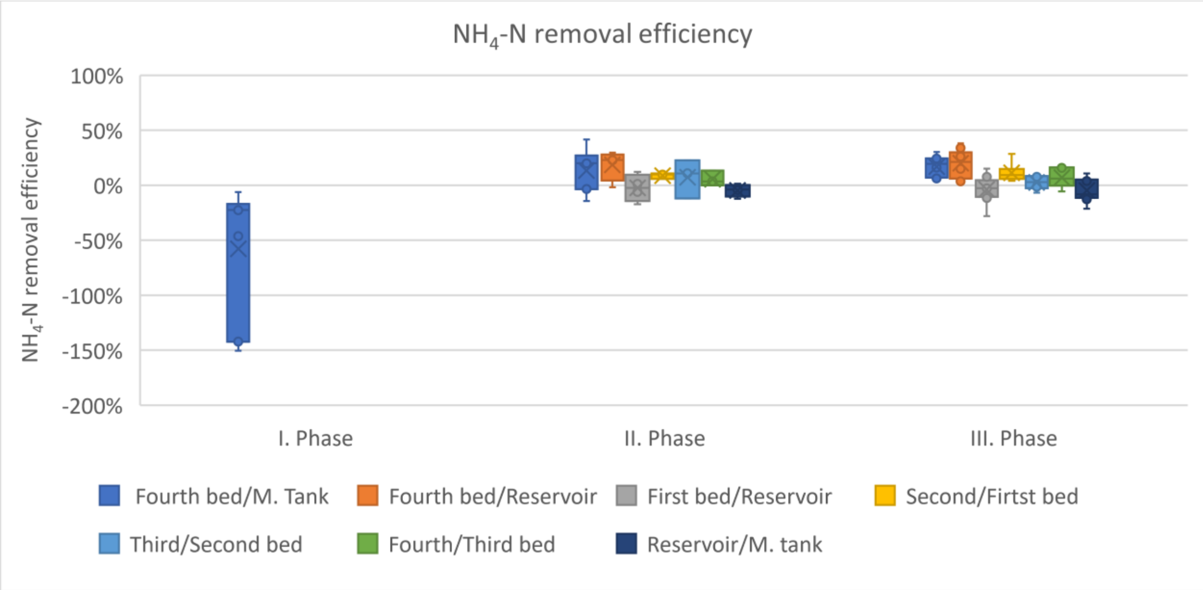
Graph 16: Individual Nitrate measurements in greywater in mixing tank, reservoir, and in the green wall beds (I. Phase n = 14, II.phase n = 29, III. Phase n = 43).



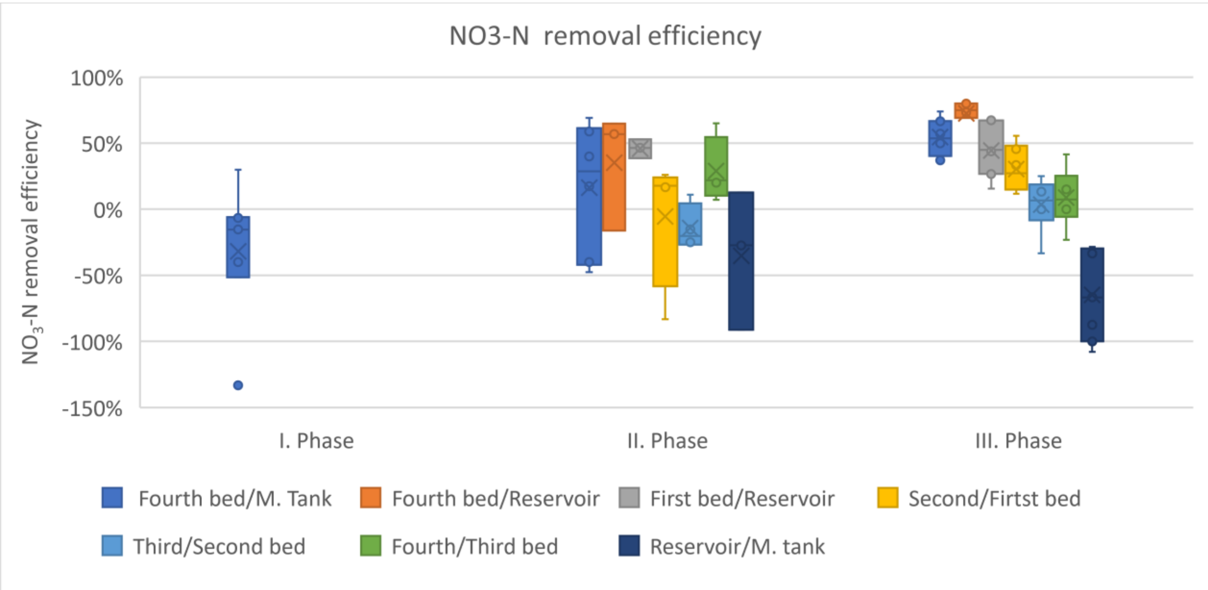
Graph 17: The box plot of Ammonium measurements in the mixing tank, reservoir, and the green wall beds according to the phases.



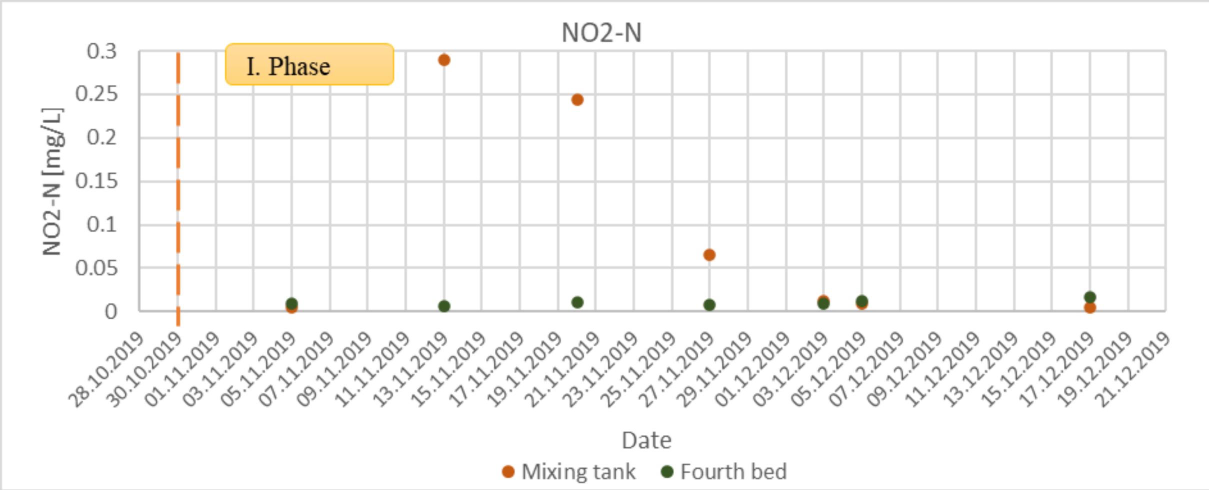
Graph 18: The box plot of Nitrate measurements in the mixing tank, reservoir, and the green wall beds according to the phases.



Graph 19: The box plot of Ammonium removal efficiency of greywater treatment between mixing tank, reservoir, and the green wall beds according to the phases.



Graph 20: The box plot of Nitrate removal efficiency of greywater treatment between mixing tank, reservoir, and the green wall beds according to the phases.



Graph 21: Individual Nitrite measurements in greywater in mixing tank, reservoir, and in the green wall beds according to the I. Phase (I. Phase n = 14).

<u>I. anoxic phase</u> included measurements only from the mixing tank and the last bed of the green wall due to estimation that almost no degradation of greywater is taking place in the mixing tank. NH_4-N mean values in the mixing tank was in average 5.15 mg/L and after the treatment in the green wall it increased to an average 6.64 mg/L (Graph 17). Whereas the initial NO_3-N concentration was in average 1.44 mg/L and after the treatment in the green wall it increased to an average 1.76 mg/L (Graph 18). Thus, there was no removal of nitrogen in general in this phase, on the contrary due to anoxic conditions ammonia was additionally released in the beds and increased the effluent concentrations (see Graph 17, Graph 19 for NH_4-N and Graph 18, Graph 20 for NO_3-N).

<u>In the II. anoxic/aerobic phase</u> with halve of the greywater concentrations the average NH_4-N influent value from the mixing tank was 3.18 mg/L. Due to aeration in the reservoir a small part was nitrified and the concentration entering the green wall was then 3.08 mg/L. The average effluent value from the fourth bed was 2.70 mg/L (Graph 17). No NH_4-N was removed from mixing tank to reservoir. In average 0.60 mg/L of NH_4-N removed from reservoir to green wall (18% efficiency) and 0.39 mg/L of NH_4-N removed from mixing tank to green wall total (13% efficiency) (see Graph 17, Graph 19 for NH_4-N).

The NO_3-N load in the mixing tank in average was 2.91 mg/L, it logically increased in reservoir to 3.53 mg/L due to ammonia nitrification to NO_3-N and after the treatment in the green wall it dropped to an average of 1.76 mg/L (Graph 18). In the II. Phase no NO_3-N was removed from mixing tank to reservoir, 0.89 mg/L of NO_3-N was removed from reservoir to green wall (35% efficiency), 1.15 mg/L of NO_3-N was removed from mixing tank to green wall total (16% efficiency) (see Graph 18, Graph 20 for NO_3-N).

<u>Lastly, in the III. anoxic/aerobic phase</u> the average NH_4-N influent value from the mixing tank was 3.74 mg/L, from the aerated reservoir was 3.92 mg/L and the average effluent value from the fourth bed was 3.12 mg/L (Graph 17). No NH_4-N was removed from mixing tank to reservoir. In average 0.80 mg/L of NH_4-N was removed from reservoir to green wall (20% efficiency) and 0.62 mg/L of NH_4-N was removed from mixing tank to green wall total (17% efficiency) (see Graph 17, Graph 19 for NH_4-N).

Whereas, the NO_3-N load in the mixing tank in average was 2.09 mg/L, in reservoir 3.01mg/L and it stayed equivalent to the former phase in average of 1.00 mg/L at effluent (Graph 18). Meaning, in the III. Phase, no NO_3-N was removed from mixing tank to reservoir, 1.64 mg/L of NO_3-N was removed from reservoir to green wall (72% efficiency) and 1.09 mg/L of NO_3-N was removed from mixing tank to green wall total (55% efficiency) (see Graph 18, Graph 20 for NO_3-N).

The removal of nitrogen is mainly performed by microorganisms through (1) nitrification, and (2) denitrification which are highly dependent on anoxic conditions and organic carbon availability (Vymazal, 2010, 2013).

Reasons for no removal in I. Phase are: (1) the wall was operating under anoxic conditions (slower process rates) and (2) ammonification of organic nitrogen under anoxic conditions. Whereas, reasons for higher removal in II. and III. Phase are (1) aerobic conditions and (2) better acclimatization of microbes.

From all the measured parameters, ammonium nitrification apart from the odours, was the reason why most of the changes have been made to the green wall system. In fact, to nitrify ammonium to nitrite and nitrate, obligate aerobe bacteria that function only in the presence of molecular O_2 needed to evolve in the substrate, since nitrification occurs only near the atmosphere, at the surface of the green wall sediment-substrate, and possibly near the roots of plants.

No additional ammonium was added to the system at the beginning of the experiment until the 21th day (19.11.2019). Already to this day all four measurements 0.85 mg/L, 1.23 mg/L at the influent (mixing tank) and at the effluent (fourth bed) 2.06 mg/L, 3.08 mg/L (Graph 15), signalled that ammonification of organic nitrogen was happening under anoxic conditions and ammonium was accumulating in the substrate. As well as ammonium, nitrate values were also higher at the effluent than at the influent in the I. Phase (Graph 15, Graph 17 for NH_4-4 and Graph 16, Graph 18 for NO_3-N). However, ammonium and nitrate efficiency values gradually improved through time, especially after air blowers and intermittent flow were applied to the green wall system in the II. Phase. This had to be made to accelerate the treatment process by increasing the levels of oxygen in the green wall beds from anoxic <0.5 mg/L to aerobic >2 mg/L.

As mentioned previously, in the I. Phase, due to the green wall operating under anoxic conditions (slower process rates), ammonification of organic nitrogen and accumulation of ammonium in the substrate, all the NH_4-N measurements were higher at the effluent than at the influent. No ammonium removal established in the I. Phase. The average reached efficiency of <u>ammonium treatment</u> between mixing tank and the fourth bed increased to 13% in the II. Phase and to 17% in the III. Phase. Whereas average reached efficiency of ammonium treatment between reservoir and the fourth bed in the II. Phase was 18%, it increased to 20% in the III. Phase (Graph 19).

Furthermore, in the beds the root zone was actively ventilated and consequently nitrification was expected, but not denitrification. Contrary to what was expected, the average reached efficiency for <u>nitrate treatment</u> between fourth bed/mix. tank increased from no removal in the I. Phase to 16% in the II. Phase and to 55% in the III. Phase. Whereas average reached efficiency of nitrate treatment between reservoir/fourth bed in the II. Phase was 35% and it increased to 72% in the III. Phase (Graph 20). High nitrate reduction suggests that predominantly anoxic conditions were established in the II., and III. Phase under the aeration pipe inserted in the middle of the beds (Figure 21), regardless the aeration. When observing the individual values of nitrate, it can be concluded that the highest detected nitrate values were usually in the reservoir with free greywater surface, which received the most dispersed oxygen due to application of air blowers in the II. Phase, which resulted in nitrification i.e conversion of ammonium to nitrate. Whereas the highest ammonium values in both II. and III. Phase were detected in the first bed. This is plausible since COD will be the first to decompose and when enough oxygen is left nitrification will follow. Nitrification in biofilms only takes place with enough dissolved oxygen and after a substantial amount of the degradable organic fraction has been removed (Bahgat *et al.*,1999).

For effective nitrification, the COD/N ratio should be kept low (<5.0) to minimize the competition between autotrophic and heterotrophic bacteria (Carrera *et al.* 2003, 2004). Therefore, first bed treated organics the most and the following second bed was able to further aerate the greywater and consequently treated the most of ammonium 9% in the II. Phase and 11% in III. Phase (Graph 19). Each next bed achieved some treatment of ammonium in most cases. However, nitrate treatment in beds varied from day to day and the efficiency at the last two beds sometimes achieved no treatment 0% (Graph 20). Last two beds were intended to treat COD, since the concentrations in the effluent were still high (>150 mg/L, see Graph 7). Therefore, aeration was not switched off. The oxygen levels in the third bed were maintained high >3–4 mg/L (Graph 12).

To achieve denitrification, aeration should be stopped to fully accomplish anoxic conditions with oxygen (DO) concentrations <0.2 mg/L at least in the third bed, which was designed for denitrification by inserting a layer of additional carbon (coconut fibres). In contrast to nitrification, generally the COD/N ratio is always kept higher than >5.0 to ensure a high denitrification rate and when lower, the external source of organic carbon is required for an efficient denitrification (Carrera *et al.*, 2004).

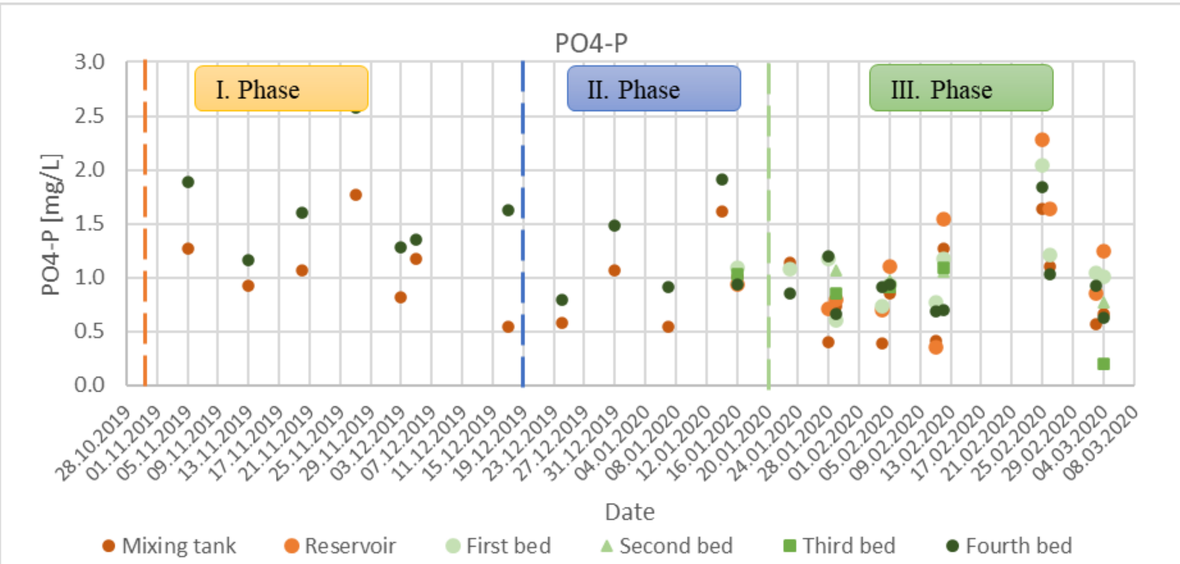
Nitrite was measured several times in the I. Phase and one time in the II. Phase. The measurements showed that there was a very low concentrations of the nitrite present in the mixed greywater usually <0.012 mg/L at the influent and about 0.011 mg/L at the effluent, so the analyses of the nitrite were abandoned later in the experiment, since they were not representative and the concentrations were negligible.

9.5 Phosphate

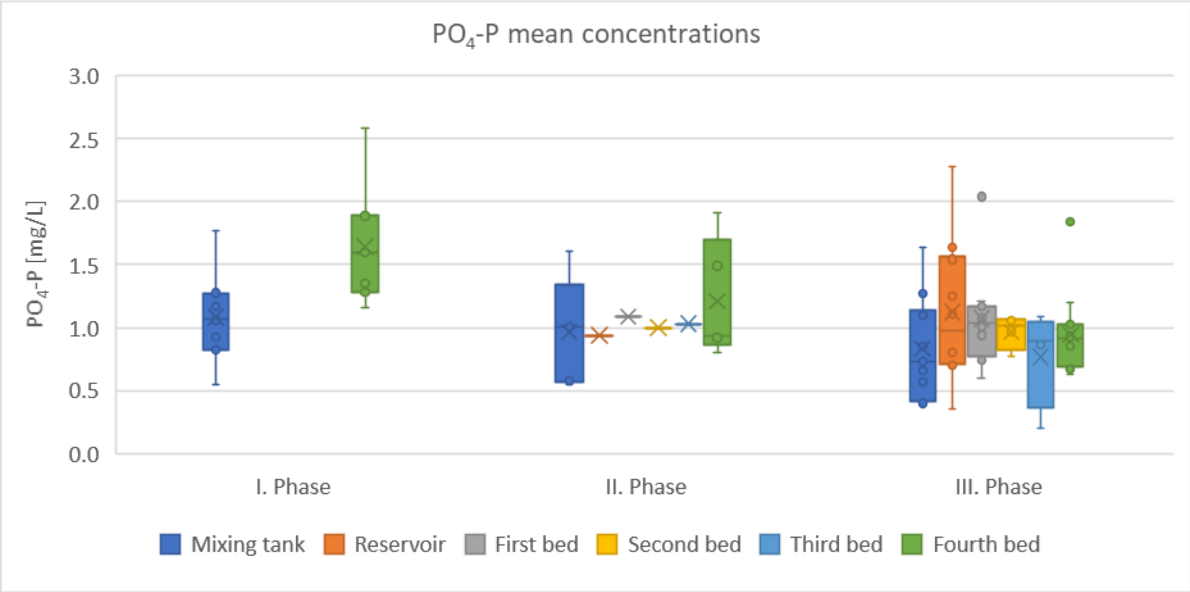
Summary of the results related to PO_4-P removal in all three phases are presented at six control points: mixing tank, reservoir, first bed, second bed, third bed and fourth bed in:

the graphs presenting PO_4-P values:

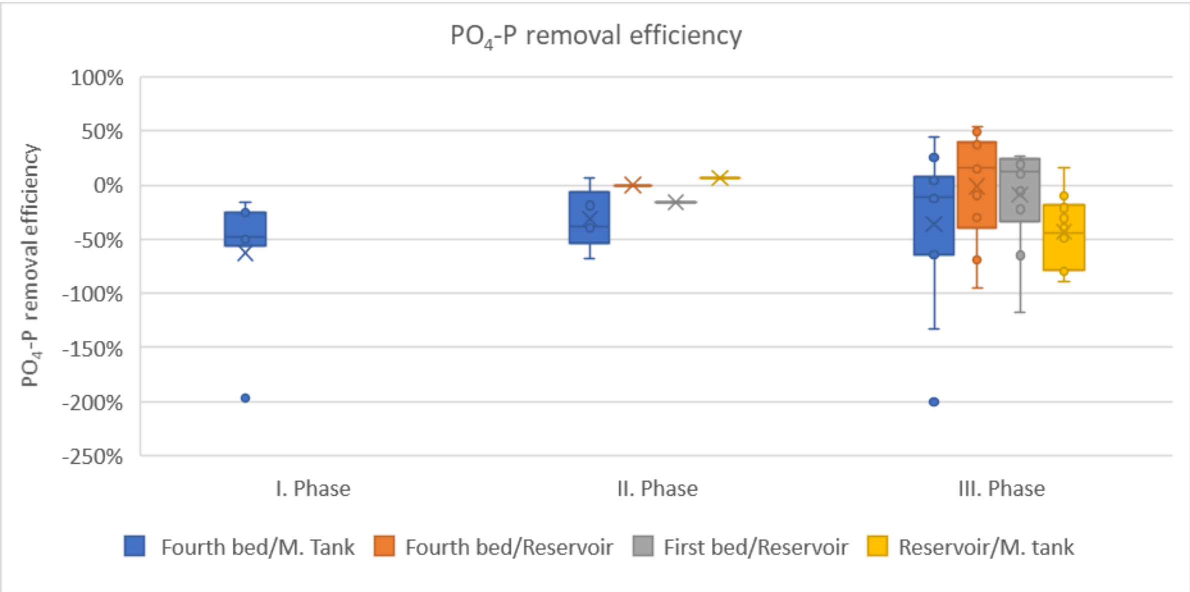
- Graph 22: Individual Phosphate measurements in greywater in mixing tank, reservoir, and in the fourth bed (I. Phase n = 14, II phase n = 14, III. Phase n = 51).
- Graph 23: The box plot of Phosphate measurements in the mixing tank, reservoir, and the green wall beds according to the phases.
- Graph 24: The box plot of Phosphate removal efficiency of greywater treatment between mixing tank, reservoir, and the green wall beds according to the phases.



Graph 22: Individual Phosphate measurements in greywater in mixing tank, reservoir, and in the fourth bed (I. Phase n = 14, II phase n = 14, III. Phase n = 51).



Graph 23: The box plot of Phosphate measurements in the mixing tank, reservoir, and the green wall beds according to the phases.



Graph 24: The box plot of Phosphate removal efficiency of greywater treatment between mixing tank, reservoir, and the green wall beds according to the phases.

<u>I. anoxic phase</u> included measurements only from the mixing tank and the last bed of the green wall due to estimation that almost no degradation of greywater is taking place in the mixing tank. PO_4-P mean values in the mixing tank was in average 1.08 mg/L and after the treatment in the green wall it increased to an average 1.64 mg/L. Meaning, no PO_4-P removal established in the I. Phase (see Graph 23, Graph 24).

<u>In the II. anoxic/aerobic phase</u> with halved greywater concentrations the average PO_4-P influent value from the mixing tank was 0.96 mg/L, from the aerated reservoir was then 0.94 mg/L and the average effluent value from the fourth bed was 1.32 mg/L. In average 0.07 mg/L PO_4-P was removed from mixing tank to reservoir (7% efficiency), no PO_4-P was removed from reservoir to green wall and no PO_4-P was removed from mixing tank to green wall total (see Graph 23, Graph 24).

Lastly, in the III. anoxic/aerobic phase the average influent value from the mixing tank was 0.83 mg/L, from the aerated reservoir was 1.12 mg/L and the average effluent value from the fourth bed was 0.94 mg/L. Meaning, no PO_4-P removal established in the III. Phase (see Graph 23, Graph 24).

Phosphorous is mainly removed by assimilation of organisms: (1) algae, (2) bacteria (Phosphorous accumulating organism, PAO - *Acinetobacter* and *Pseudomonas*), (3) plants); and by (4) precipitation of minerals (e.g., Ca, Fe, Al, Mg).

Reasons for no phosphorous removal in any of the phases: (1) no plants at all (I. and II. Phase) or no plants with high phosphorous uptake (III. Phase), (2) not a growing season, (3) substrate without containing specific minerals (e.g., Ca, Fe, Mg, Al) for precipitation, (4) no PAO microbes present and required anaerobic conditions for their establishment.

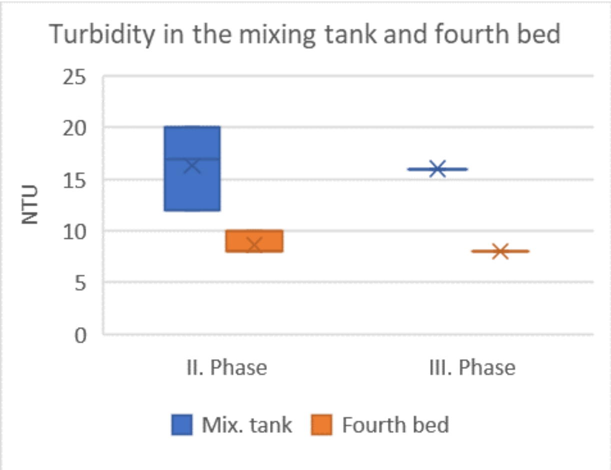
In average, no phosphorous treatment was achieved in neither of the three phases. In contrast, adsorption and leaching of phosphorous was detected since the values were usually higher at the effluent than at the influent. The inlet values of phosphate were usually around 1 mg/L in the mixing tank. The low phosphorus content, on the other hand, can be related to the fact that most washing detergents no longer contain phosphates, because of their toxicity, nutrient pollution and consequently algae blooms (Jefferson *et al.*, 2004) since, the average effluent values were 1.64 mg/L, 1.32 mg/L and 0.94 in the I., II. and III. Phase. Lower phosphorus removal compared to nitrogen removal has similarly been reported in several wetland systems (Brix and Arias, 2005; Stefanakis and Tsihrintzis, 2012). In the study conducted by Tanner (1996), the average rate of plant uptake was greater for nitrogen than phosphorus, with nitrogen showing a significant linear relationship to plant biomass, a trend which was not that apparent for phosphorus. Also, the plants in this green wall setup were not particularly selected to have a higher nutrient uptake and the experiment took place during the winter season when plants do not take up as many nutrients. While nitrogen removal occurs mostly through biological avenues and is therefore strongly influenced by the presence of plants (Brix, 1997), microorganisms, adsorption onto the filter media and precipitation reactions with minerals present in the filter media have been shown to be more significant retention pathways for phosphorus compared to plant uptake in vertical wetland systems (Dzakpasu *et al.,* 2015; Tanner, 1996). Moreover, taking TW as a reference, it is possible to predict that the precipitation and adsorption of phosphorus are higher under saturated conditions because of the low fluctuation in redox potential. With saturated zone and anoxic conditions established in the green wall, conditions for precipitation and adsorption were assured. On the other hand, phosphorous is removed by precipitation of specific minerals (e.g., Ca, Fe, Mg, Al) with sorption being the general removal mechanism. The sand filter media silica sand (Si_2O) and perlite (SiO_2) used in this study may possibly have a low phosphorus sorption index, which could explain the lower phosphorus removal efficiency compared to nitrogen. For future attempts substrate richer in minerals, such as limestone with higher sorption index is recommended. Phosphorous is also removed by assimilation of Phosphorous accumulating organism, PAO - *Acinetobacter* and *Pseudomonas* under certain conditions, namely by alternating first anaerobic (no O_2 and no NO_3) and then aerobic conditions. The first were never achieved at least NO_3 was present in the wall. Plus, enough volatile fatty acids (VFA) are needed in the anaerobic conditions. Moreover, those bacteria need to be added manually, which was not the case in this experimental setup and anaerobic conditions were most likely not established. These conditions can be easier achieved in more controlled technologies than TW. Since the values of PO_4-P in the treated synthetic greywater were already very low (< 2 mg/L), these results do not affect the reuse potential of the green wall effluent (see Table 4).

9.6 Turbidity

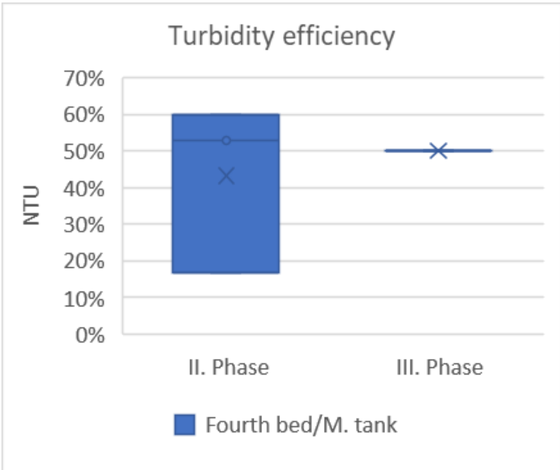
Summary of the results related to Turbidity removal in II. and III. Phase is presented at two control points: mixing tank and fourth bed in:

the graphs presenting PO_4-P values:

- Graph 26: The box plot of Turbidity measurements in the mixing tank and fourth bed according to II. and III. Phase.
- Graph 25: The box plot of Turbidity removal efficiency in the mixing tank and fourth bed according to II. and III. Phase.



Graph 26: The box plot of Turbidity measurements in the mixing tank and fourth bed according to II. and III. Phase.



Graph 25: The box plot of Turbidity removal efficiency in the mixing tank and fourth bed according to II. and III. Phase.

The turbidity values were informatively measured at 450 nm 3 times in II. Phase and 2 times in the III. Phase. The inflow turbidity mean value at the mixing tank in the II. Phase was 16.3 NTU and dropped to about 8.7 NTU at the effluent in the fourth bed (43% efficiency). The inflow turbidity mean value in the III. Phase was 16.3 NTU and it dropped to about 8.7 NTU at the effluent (50% efficiency) (Graph 25, Graph 26).

Turbidity is mainly removed by (1) filtration and (2) sedimentation.

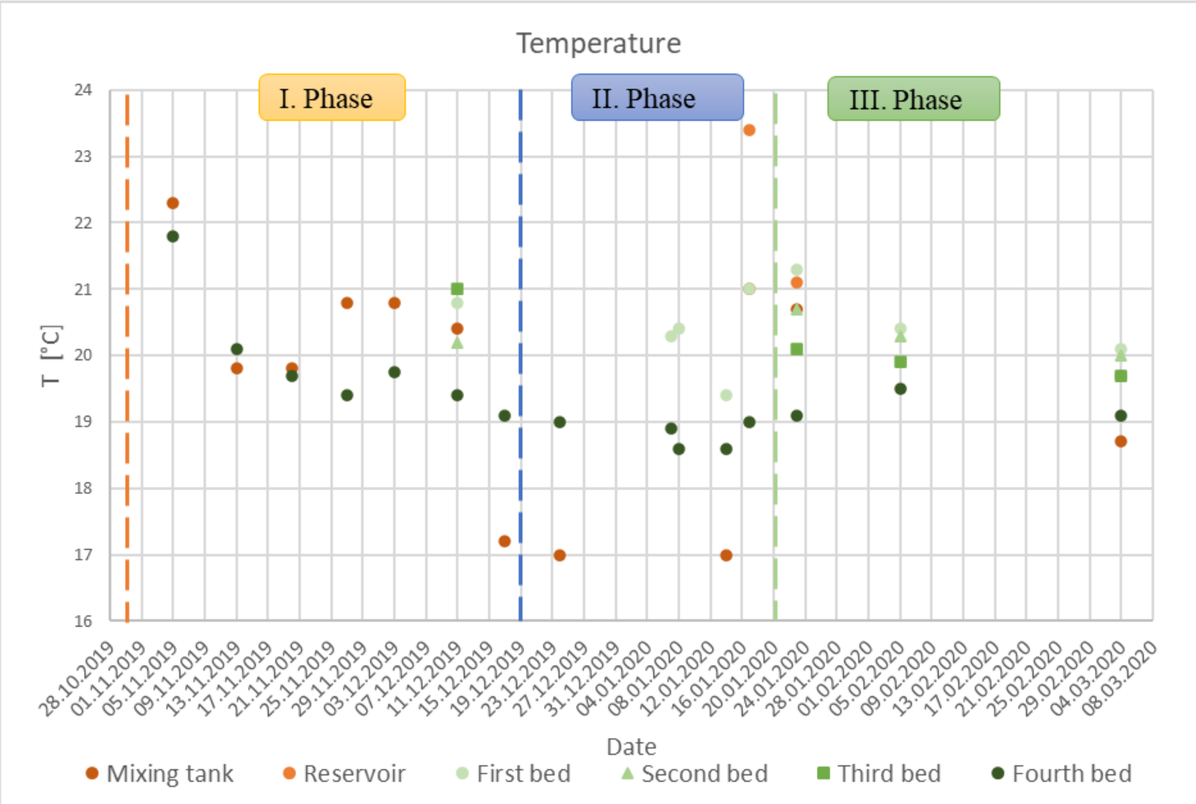
The US's turbidity criteria standard for unrestricted reuse is 2 NTU (restricted reuse does not have a turbidity requirement) and criteria for agricultural reuse in EU is 5 NTU. Turbidity is likely attributed to the enhanced contact time with the range of media types found in these systems, particularly the lower, finer layers. (Gerba *et al.,* 1999; Li *et al.,* 2009). To improve turbidity aeration in the beds should be switched off to encourage sedimentation in the substrate, the flow should be decreased or by adding a finer substrate such as coco coir which was reported to be successful at lowering turbidity by other studies (Prodanovic *et al.*, 2020, Fowdar *et al.*, 2017).

9.7 Temperature

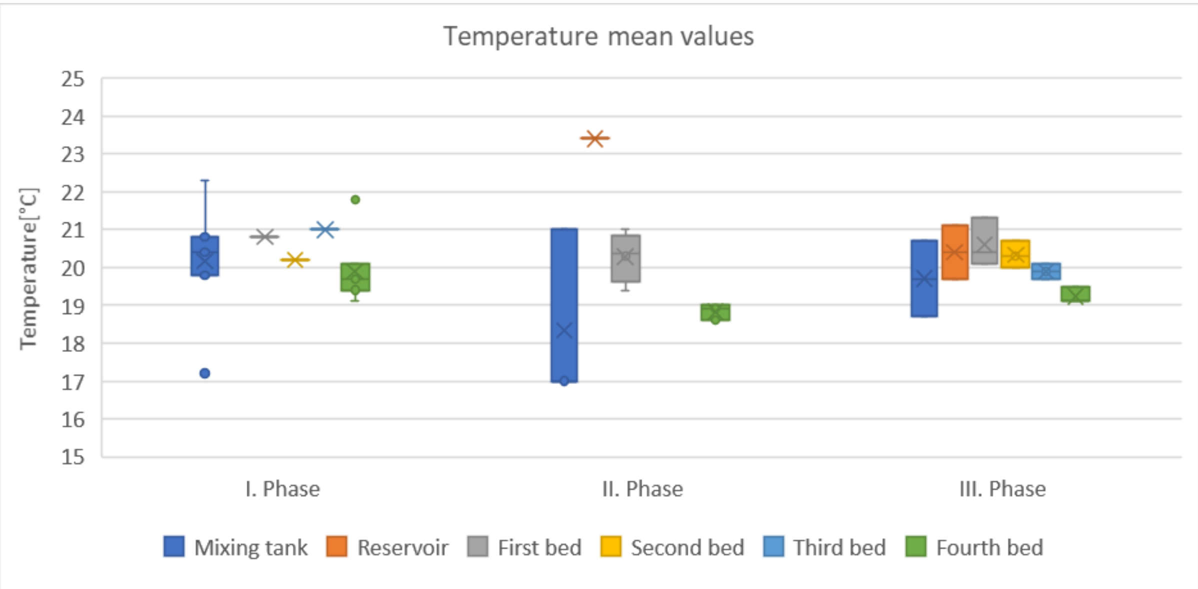
Summary of the results related to Temperature in all three phases are presented at six control points: mixing tank, reservoir, first bed, second bed, third bed and fourth bed in:

the graphs presenting Temperature values:

- Graph 27: Individual Temperature measurements in greywater in mixing tank, reservoir, and in the green wall beds (I. Phase n = 17, II phase n = 13, III. Phase n = 16).
- Graph 28: The box plot of Temperature measurements in the mixing tank, reservoir, and the green wall beds according to the phases.



Graph 27: Individual Temperature measurements in greywater in mixing tank, reservoir, and in the green wall beds (I. Phase n = 17, II phase n = 13, III. Phase n = 16).



Graph 28: The box plot of Temperature measurements in the mixing tank, reservoir, and the green wall beds according to the phases.

In the I. Phase temperatures were measured in the mixing tank, first bed, second bed, third bed and fourth bed. The measured mean temperatures in the same order were: 20.16 °C, 20.80 °C, 20.20 °C, 21.00 °C and 19.89 °C (Graph 28).

In the II. Phase temperatures were measured in the mixing tank, reservoir, first bed, and fourth bed. The measured mean temperatures in the same order were: 18.33 °C, 23.40 °C, 20.28 °C and 18.82°C (Graph 28).

In the III. Phase temperatures were measured in the mixing tank, reservoir, first bed, second bed, third bed and fourth bed. The measured mean temperatures in the same order were: 19.70 °C, 20.40 °C, 20.60 °C, 20.33 °C, 19.90 °C and 19.23 °C (Graph 28).

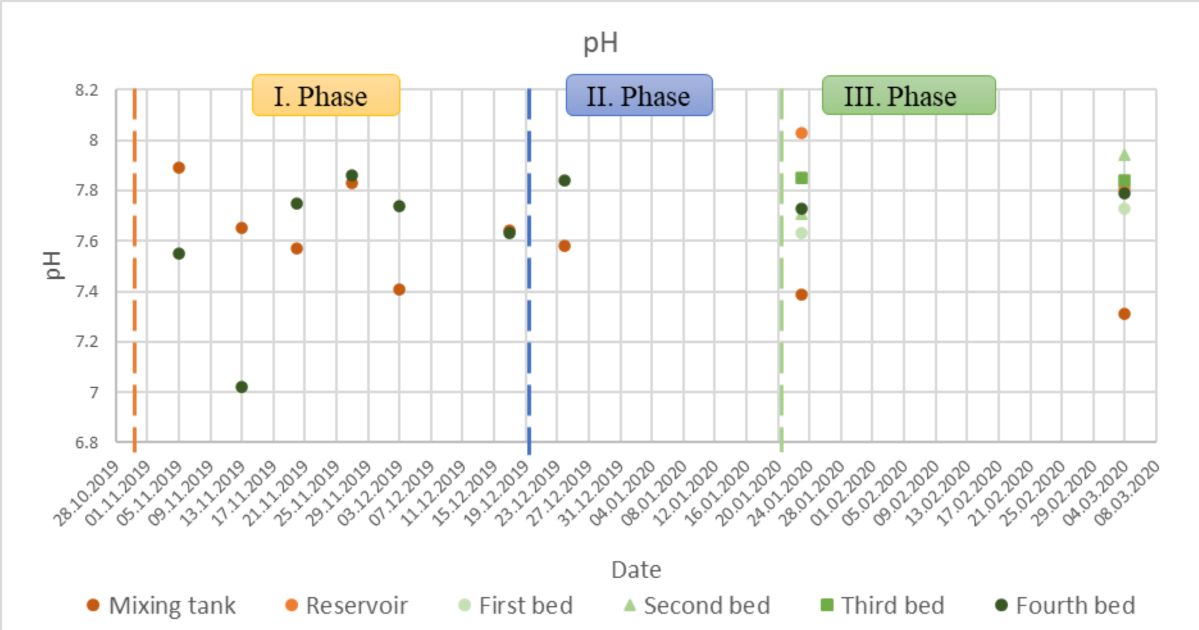
Temperature in the mixing tank varied depending at what time the measurement was taken. After the filling up the mixing tank with fresh sanitary water and adding the concentrate of cleaning products to prepare synthetic greywater, its temperature was low due to the water distribution pipes, buried under the ground level, which maintain the temperature of water around 10–12°C. Therefore, the temperature was measured other days than on the greywater filling days and was ranged from 17 to 22.3°C. Another effect on the water temperature had the weather. The tank was positioned near the windows and could be influenced by the sun. To prevent greywater from warming up and the algae growth, it was covered with the protective canvas. The temperatures in the green wall beds however did not fluctuate so much. Nevertheless, they differentiated among each other, since the temperature at the bottom beds were slightly lower than on the upper beds for a difference of 1–2°C. The reason behind temperature difference between beds could be the normal vertical distribution of air where the cooler air stays at the bottom and the warmer air rises upwards. Also, the green wall was positioned at the building's entrance door, which were being opened and closed throughout the day. Again, the weather might have influenced the greywater temperature, especially lowering them, since experiment was happening during the winter months. Nevertheless, frequent opening of the door was quite welcomed since it reduced the unpleasant odours and humid by letting the outside fresh air into the building. The air movement around the foliage is important to help prevent fungal growth (Pradhan *et al.,* 2019; Carpenter, 2014).

9.8 pH

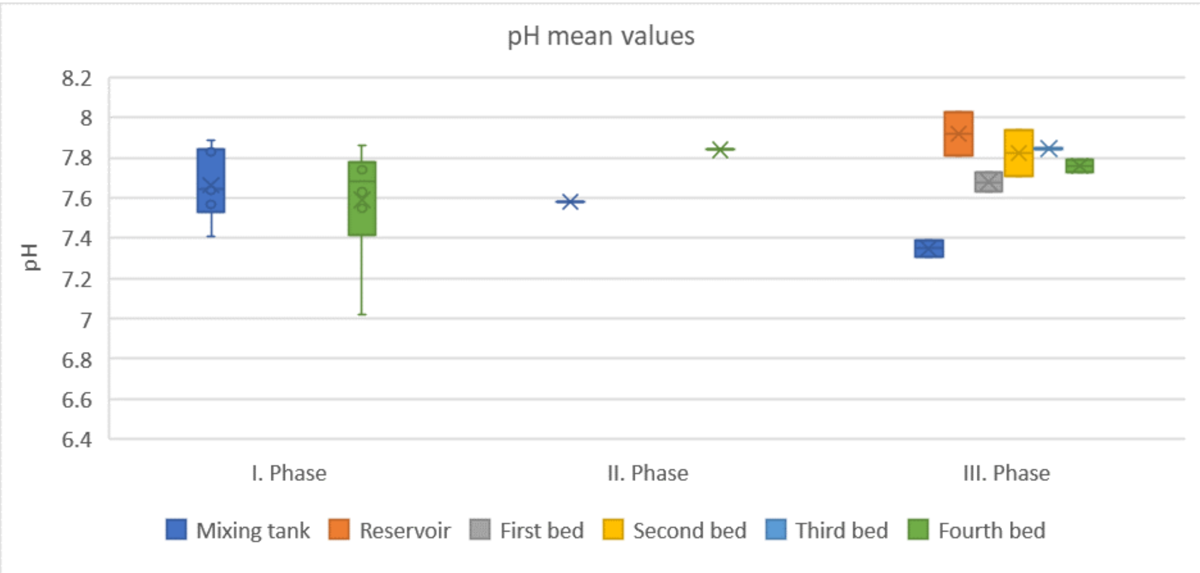
Summary of the results related to pH in all three phases are presented at six control points: mixing tank, reservoir, first bed, second bed, third bed and fourth bed in:

the graphs presenting pH values:

- Graph 29: Individual pH measurements in greywater in mixing tank, reservoir, and in the green wall beds (I. Phase n = 12, II phase n = 2, III. Phase n = 12).
- Graph 30: The box plot of pH measurements in the mixing tank, reservoir, and the green wall beds according to the phases.



Graph 29: Individual pH measurements in greywater in mixing tank, reservoir, and in the green wall beds (I. Phase n = 12, II phase n = 2, III. Phase n = 12).



Graph 30: The box plot of pH measurements in the mixing tank, reservoir, and the green wall beds according to the phases.

In the I. Phase pH was measured in the mixing tank and fourth bed. The measured mean pH values in the same order were: 7.67 and 7.59 (Graph 30).

In the II. Phase pH was measured in the mixing tank and fourth bed. The measured mean pH values in the same order were: 7.58 and 7.84 (Graph 30).

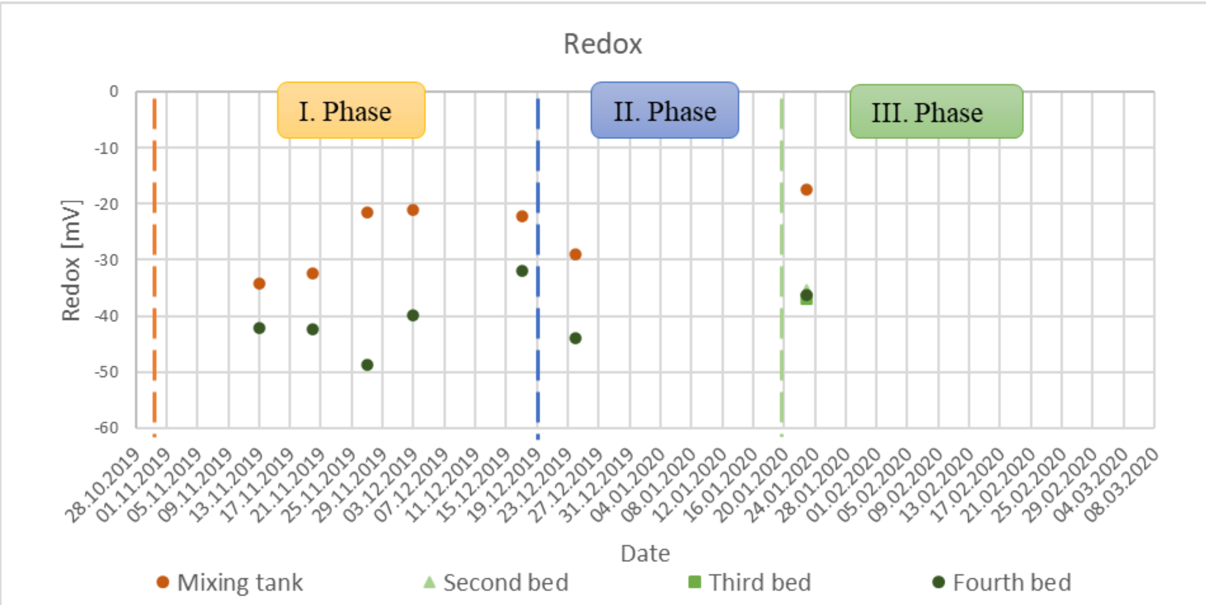
In the III. Phase pH was measured in the mixing tank, reservoir, first bed, second bed, third bed and fourth bed. The measured mean pH values in the same order were: 7.35, 7.92, 7.68, 7.83, 7.85 and 7.76 (Graph 30).

Overall, not much difference according to pH values were detected during the experiment. The values of the mixing tank and the beds ranged around neutral 7.6 pH.

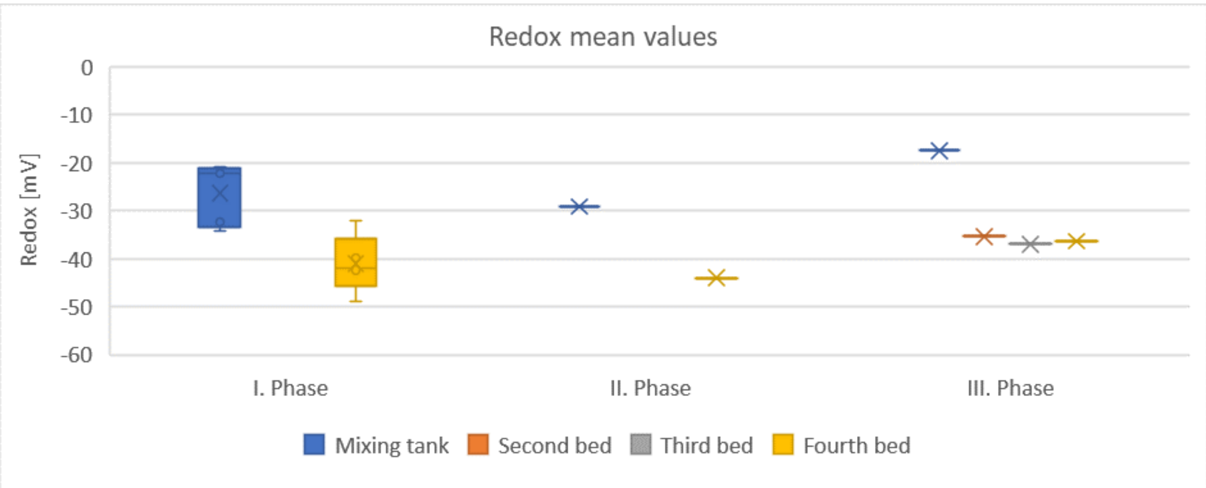
9.9 Reduction-oxidation potential (redox)

Summary of the results related to redox in all three phases are presented at five control points: mixing tank, first bed, second bed, third bed and fourth bed in the graphs presenting redox values:

- Graph 32: The box plot of redox measurements in the mixing tank, reservoir, and the green wall beds according to the phases.
- Graph 33: Individual EC measurements in greywater in mixing tank, reservoir, and in the green wall beds (I. Phase n = 10, II phase n = 2, III. Phase n = 12).



Graph 31:Individual redox measurements in greywater in mixing tank, reservoir, and in the green wall beds (I. Phase n = 10, II phase n = 2, III. Phase n = 3).



Graph 32: The box plot of redox measurements in the mixing tank, reservoir, and the green wall beds according to the phases.

In the I. Phase redox was measured in the mixing tank and fourth bed. The measured mean redox values in the same order were: -26.28 mV and -41.04 mV (Graph 32).

In the II. Phase redox was measured only once after the dilution in the mixing tank and fourth bed. The measured redox values in the same order were: -29.1 mV and -44 mV (Graph 32).

In the III. Phase redox was measured only once in the mixing tank, first bed, second bed, third bed and fourth bed. The measured redox values in the same order were: -17.5 mV, 31.3 mV, -35.4 mV, -37.0 mV, and -36.3 mV (Graph 32).

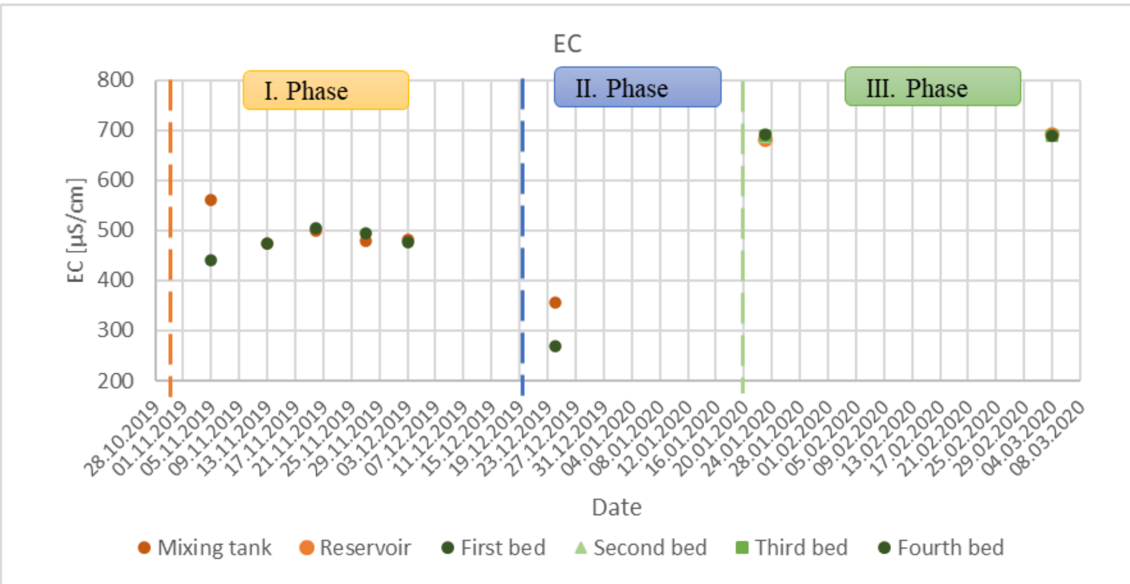
Redox potential was below 0 mV at the influent and at the effluent, suggesting anoxic conditions in the greywater. However, greywater at the effluent appeared with more negative (overall average -41 mV) than redox values than at the influent (overall average -25 mV).

9.10 Electrical conductivity (EC)

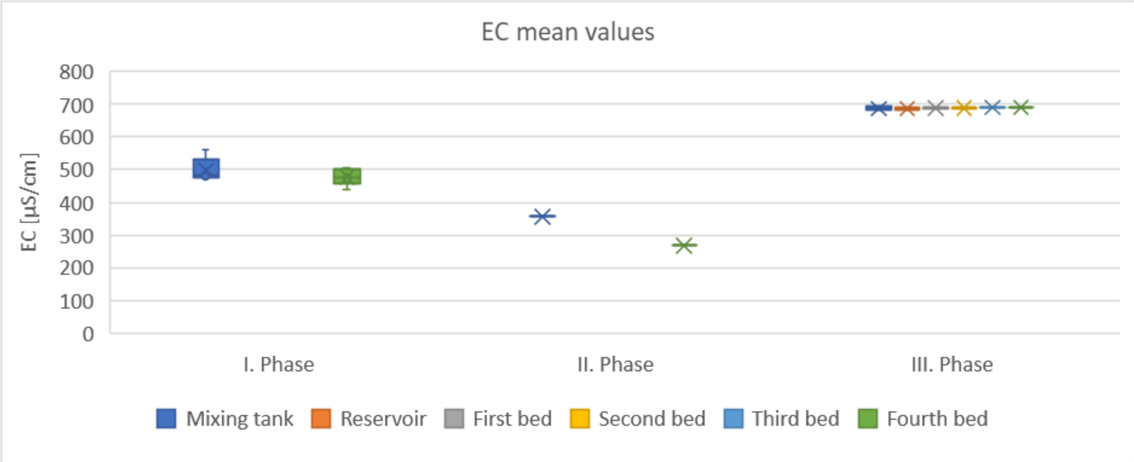
Summary of the results related to EC in all three phases are presented at six control points: mixing tank, first bed, second bed, third bed, and fourth bed in:

the graphs presenting EC values:

- Graph 33: Individual EC measurements in greywater in mixing tank, reservoir, and in the green wall beds (I. Phase n = 10, II phase n = 2, III. Phase n = 12).
- Graph 34: The box plot of EC measurements in the mixing tank, reservoir, and the green wall beds according to the phases.



Graph 33: Individual EC measurements in greywater in mixing tank, reservoir, and in the green wall beds (I. Phase n = 10, II phase n = 2, III. Phase n = 12).



Graph 34: The box plot of EC measurements in the mixing tank, reservoir, and the green wall beds according to the phases.

In the I. Phase EC was measured in the mixing tank and fourth bed. The measured mean EC values in the same order were: -26.28 µS cm^{-1} and -41.04 µS cm^{-1} (Graph 34).

In the II. Phase EC was measured only once after the dilution in the mixing tank and fourth bed. The measured EC values in the same order were: -29.1 µS cm^{-1} and -44 µS cm^{-1} (Graph 34).

In the III. Phase EC was measured only once in the mixing tank, first bed, second bed, third bed and fourth bed. The measured EC values in the same order were: -17.5, 31.3, -35.4, -37.0, and -36.3 µS cm^{-1} (Graph 34).

To conclude, electrical conductivity (EC) was more than 400 µS cm^{-1} at the influent, the main sources being the salts included in soaps, detergents, washing powders and, the tap water itself with about EC of 300 µS cm−1. However effluent EC was mostly not significantly higher from the influent EC, as was also observed by Szota *et al.* (2015) in their study on stormwater biofilters receiving influent of salinity >10 mS/cm (Fowdar *et al.,* 2017). Higher effluent EC than in the influent, can occur due to water loss (evapotranspiration), plant uptake and release (Albalawneh *et al.*, 2016) or salt release from body care products and other products (Zraurig *et al.* 2019). The difference between lower EC values during the I. Phase than the II. Phase was due to the intended dilution of greywater.

9.11 Determining design parameters and proposal about future additions to the pilot system design

During the pilot experiment several obstacles needed to be overcome in order to achieve a properly operating indoor green wall greywater treatment system with horizontal flow. Moreover, to improve the design and treatment efficiency, a list of remarks and recommendations for future work are presented in this section.

The experimental period of the current work lasted for a very short time, only 4 months in comparison to other studies. The start-up phase lasted only one month in comparison to other studies which dedicated up to 3 months for a system to adapt and up to two years for the whole experimental trial (see Table 14). To have a better insight on system adaptation, analysing, and measuring the type, and growth of microorganisms is recommended. Therefore, for further research, experimental time of such systems should be considered. The findings in this study are based on a relatively small number of samples, all of which may be classified under the start-up period for the treatment system. An extended study, covering all seasons throughout the year, would provide more information regarding the actual treatment capabilities. Nevertheless, the greywater treatment system was shown to significantly reduce nutrient concentrations in the effluent water.

Organic load and substrate medium volume are important parameters for green wall design. This research indicated that OLR of 68 g COD/m^2d is too high as applied at the initial phase. With a reduction to 33 g COD/m^2d might have been the bare maximum a system could treat without the intense odour (due to anoxic conditions). To improve treatment, HRT can be increased and HLR decreased by altering the frequency of flushes (flooding and draining cycles). In current experiment the HRT was set to 18 hours which was too low for the OLR (33 g COD/m^2d) in this study. Previous studies used HRT of at least 24 hours for similar OLR such as a study by Zraunig *et al.* (2019) (see Table 14). However, treatment should improve over the operational time (after 4 months) despite the OLRs used in this study. HLR 0.11 m/d for all beds area combined (0.44 m/d per one bed) was recognised similar to other studies and higher than the USEPA (2002) recommendations on HLR as 0.04–0.08 m/d of intermittent sand filters for treating full strength domestic wastewater. The latter is feasable since greywater is less polluted than wastewater and can therefore be treated in larger amounts. To determine a perfect design ratio between OLR, HRT and HLR further research is needed. Also, it is possible to design the system with circulation of greywater until it reaches the desired treatment efficiency.

Next, fully anoxic conditions are not suitable for indoor green wall application since it can result in producing unpleasant odours. Therefore, providing aeration is recommended. This can be archived with intermittent loading with at least one resting hour in between. This allows the air compressed during previous flooding to be uniformly distributed throughout the media. A test of hydraulics after one feeding would tell exactly what the suitable duration time is between the feedings. In other words, the actual HRT needs to be measured, i.e. the time needed for greywater to stop flowing from the start to the end of the green wall. Furthermore, aeration can be achieved with air blowers or by designing hybrid green wall with vertical and horizontal flow. It is necessary to provide both aerobic and anoxic conditions in order to achieve nitrification and denitrification which might be best achieved with hybrid systems. Hybrid systems include the advantage of combining horizontal flow and vertical flow, providing different redox environments, which can significantly improve the conditions needed for nitrification and denitrification processes, adsorption and precipitation of phosphorus and removal of organic matter. The vertical flow brings aerobic conditions needed to remove ammonia–N by nitrification/volatilization and BOD by bacterial oxidation, while horizontal flow brings anoxic conditions which increases the removal of nitrogen and phosphorus, through denitrification and precipitation (Vymazal 2007, 2013).

Hybrid flow was considered in the designing phase of this prototype. Therefore, at the bottom of each bed 4 holes were drilled with a purpose of establishing a vertical flow or hybrid flow for future research (see Figure 18). In the current experimental setting, aeration should be switched off in the third bed to achieve fully anoxic conditions which are typical for horizontal flow in the first place and by this, improving the denitrification. According to the oxygen concentration, it should be emphasised that it was difficult to determine the exact conditions established along the green wall beds section. Therefore, it is proposed to take samples also from the lower-middle of the beds in future attempts. Also, an investigation of aeration conditions using dissolved oxygen measurements and gas tracer studies along the substrate section would be valuable information in order to characterize the effect of the green wall design.

In the green wall pilot system, a mix of perlite (2–6 mm), silica sand (2–6 mm), and coconut fibres were selected for a substrate. It provided suitable hydraulic conductivity in second, third and fourth bed, whereas in the first bed intense clogging was noticed. It is recommended that larger diameters of gravel and sand are selected for the first bed or at least for the inlet area. Kadlec (2009) reported that in TWs with horizontal flow, clogging of the inlet area normally causes a great reduction of hydraulic conductivity. Castellar Da Cunha *et al.* (2018) suggested the design with the implementation of an inlet layer filled with gravel (10 mm < Ø < 16 mm) in order to minimize inlet clogging effects. In addition, to avoid intense clogging, increasing the slope for 1% can balance the greywater level in the beds and spread water further from the inlet of the bed. This is beneficial to the plant roots in a way of lowering saturated zone at the start of the bed, so the greywater does not reach the surface of the substrate and floods the plant roots.

According to the used substrate material, a shortage of minerals, i.e. Ca, Fe, Mg or Al for removal of phosphorous, was recognised. On the other hand, filters with horizontal flow require large amounts of organic carbon to promote denitrification (if nitrogen is present) and at the same time are quite efficient for the adsorption and precipitation of phosphorus if the substrate is rich with minerals (Vymazal, 2007). Therefore, a layer composed by a mixture (1:1) of mineral and organic media rich is recommended, such as coconut coir that got many recommendations due to efficient organics, solids, turbidity, nitrogen, and phosphorus removal from several researchers previously mentioned. Coconut coir has also been previously used as a slower media which could be applied in the last two beds of the pilot green wall designed in this work where the potential of clogging is minimal. In this way, turbidity at the outflow could reach as low as 2 NTU, which meets the standards for unrestricted reuse. However, adding organic media into the substrate is feasible only at times when nitrogen concentrations are high, which with greywater usually is not the case. Lastly, it would be interesting not only focusing on indoor-ornamental plants that are keen to wet substrate but also into choosing different plants with higher nutrient uptake. Therefore, a further research on the indoor tropical/wetland plants that are known for larger nutrient uptake is proposed.

10 CONCLUSION

This study presents the design, implementation, and a thorough assessment of a green wall technology for greywater treatment. It demonstrates that greywater can be a promising reusable source of water for non-potable uses. Although not the subject of experiment here, the study indicates how thermal energy for preheating domestic cold water can be simultaneously employed in the presented system.

The experimental setup consisted of four green wall beds and four water tanks: a mixing tank for greywater preparation, a heat exchanger (not in function), a reservoir for greywater aeration and water supply, and a greywater collecting tank. The wall was operated in three phases that describe three different operational conditions. In Phase I., the greywater used was heavily loaded; no aeration was used and no vegetation was planted. In II. Phase the greywater load was halved and aeration was introduced, and in III. Phase vegetation was planted. Chemical and physical parameters were monitored throughout the experiment. Measurements were taken at six control points: the mixing tank, reservoir, and four green wall beds. The main measured physical parameters were: greywater flow entering the first bed, the temperature, oxygen, pH, redox, and electrical conductivity of greywater in all the tanks and beds. The main chemical parameters regularly observed were COD, BOD, NH_4-N, NO_3-N, and PO_4-P. NO_2-N, and turbidity were measured irregularly. The results show the following:

1. Sufficient removal efficiency was achieved in III. Phase: >70% COD, 74% BOD, 20% NH_4-N and 72% NO_3-N removal efficiency. Average outflow concentrations from the green wall in III. Phase were 88 mg COD/L, 15 mg BOD/L, 3 mg NH_4-N/L, and 1 mg NO_3-N/L.

2. The wall achieved these results operating with organic loading rate of 33 g COD/m^{2}d and 7 g BOD/m^{2}d (in III. Phase). Phosphorous was not planned to be treated in the pilot green wall, wherefore no phosphorous treatment was achieved in either of the three phases.

3. The effluent from the green wall satisfied some reuse quality standards (Table 33), i.e. the BOD concentration in the effluent from the green wall designs satisfied the existing guidelines for BOD urban reuse of most countries that already have them. Some effluent concentration satisfied US's standards, which for example require <10 mg BOD/L for unrestricted urban reuse (non-potable applications with uncontrolled public access). Certainly, the effluent concentrations satisfied the US's guidelines for environmental reuse requiring <30 mg/L in terms of BOD and for certain reuses under Italy's standards in terms of COD requirement requiring <100 mg/L. Regarding US's standards on turbidity, the results (8.7 NTU) were close to meeting the guideline for unrestricted urban reuse (<2 NTU) and agricultural reuse (<5 NTU). The results of this study demonstrate that innovative and aesthetically pleasing vegetated green walls can be designed for treating greywater and are a promising technology not only for an outdoor but also for an indoor, on-site, urban greywater treatment at the household scale.

Table 33: Effluent concentrations from III. Phase in comparison to some reuse quality standards

	US (USEPA, 2012)	**Western Australia (GWA, 2010).**	**Cyprus (KDP269/2005, KDP 772/2003)**	**EU (2018/0169 (COD))**	**Italy (DM 185/2003)**	**Green wall in Master's Thesis (2021)**
BOD [mg/L]	unrestricted urban reuse <10 environmental reuse **<30**	irrigation **<20**		<10		15
COD [mg/L]			<70		**<100**	88
Turbidity [NTU]	<2			<5		8.7

4. According to the results following recommendations can be proposed. This thesis recommends following design parameters for green wall greywater treating system and approaches, such as duration of a start-up phase, hydraulic retention time (HRT), hydraulic loading rate (HLR), organic loading rate (OLR), bed dimensions, substrate selection and oxygen conditions in the substrate.

Although the green wall concept was not entirely verified, the end treatment results showed great potential for improvement. Further work can focus on heat recovery from greywater; microbial activity, the types of microorganisms both present in greywater and accountable for its biological treatment, optimizing plant selection with species that have a higher nutrient uptake, good acclimatisation to wetlands, and high levels of adaptation and survival in greywater. Other issues include the optimisation of a substrate selection that is both minerally and organically rich with grain sizes that facilitate phosphorous removal, denitrification, improved hydraulic conductivity through the removal of organic matter and suspended solids to reduce clogging, the optimisation of oxygen conditions by combining horizontal flow with vertical and by optimizing the intermittent feedings. These areas show a great opportunity for collaboration between wastewater treatment engineers, mechanical engineers, botanists, microbiologists, and biochemists.

11 SUMMARY

Rapid urbanization in recent decades and climate change have led to a negative effect on natural ecosystems as a consequence of continuously increasing water and energy demand (UNESCO, 2015, Strungaru *et al.,* 2015). Every day there is less available clean water on Earth. In fact, world demand for freshwater will increase by 55% between 2000 and 2050 (OECD, 2012), whereas global primary energy demand is expected to increase at an annual rate of 1.3% until 2035, with the highest increase in India at an annual rate of 2.7% already today (International Energy Outlook, 2017). Thus, the demand for applying new and alternative water and energy sources, such as reclaiming water and heat recovery from wastewater, is increasing (Garnier *et al.,* 2015; Frijns *et al.*, 2013; Ravichandran *et al.*, 20202). Heat exchangers and heat pumps (or a combination of both) can be used for extracting heat energy from wastewater (Arnell *et al.*, 2017) and consequently, saving energy for residential water heating, which accounts for 4–6% of the total national energy demand in developed countries.

Current work is focused specifically on separating greywater (GW) from wastewater, on treating and reusing it for secondary purposes, and on heat recovery. Generally, GW is characterized as a less polluted household wastewater in industrialised countries, discharged from dishwashers, showers, sinks, baths, and washing machines. It comprises the entirety of household wastewater except for wastewater from toilets (Department of Health Western Australia, 2010; Environment Agency, 2011; Eriksson *et al.*, 2002; World Health Organization 2006; Friedler and Hadari 2006). Treated GW can be reused indoors for flushing toilets or washing clothes, and for outdoor applications such as irrigating lawns on college campuses, athletic fields, cemeteries, parks, golf courses, domestic gardens (Okun, 1997), for washing vehicles and windows, fire protection, boiler feedwater, and concrete production (Okun, 1997; Santala *et al.,* 1998). There are many advantages that comes with GW reuse. For example, the reuse of GW lowers the total costs for wastewater handling, as the load of water being processed in the treatment plants reduces (Eriksson *et al.*, 2002). Greywater reuse can be applied to hotels and other densely populated places with large buildings, where water demand and greywater production is highest. Furthermore, the fast urbanization leads to changes in urban landscapes by making cities taller and "grey" rather than "green". As a consequence, the replacement of vegetation and natural areas with non-natural spaces built out of concrete, bricks, and asphalt has led to changes in the dynamics of urban ecosystems (Shooshtarian *et al.,* 2018). In fact, changes in urban geometry and use of impervious materials can lead to radiation trapping and ventilation blockage, which results in increased temperatures, recognised as heat island effect (Palme *et al.,* 2016). Therefore, water management and technologies that can treat and reuse wastewater as an alternative resource of water and energy, in addition to having a cooling effect on buildings, are becoming significantly important (Ghaitidak and Yadav, 2013; Li *et al.,* 2009; WWDR, 2015).

Similarly to treatment wetlands, an idea of a potential alternative wastewater treatment system exists, recognised under various terms such as green walls, living walls, wet walls, wet facades, etc. (Medl, Stangl and Florineth, 2017), which has not been utilised sufficiently yet. Green walls can provide acoustic comfort, create ecologically biodiverse habitats, act as insulation, purify air, and even be used as wastewater treatment plant technology. Such technology is the main focus in this master's thesis, in which the design parameters for such a system are explored. Furthermore, in the current work, various heat recovery systems were overviewed and the most suitable ones were proposed for further studies on GW in combination with green walls. A centralized GW heat recovery system with submerged spiral coiled pipe was recognised as the best type of heat exchanger to be combined with green walls for GW treatment.

A pilot green wall was designed and experimental work was done for determining its GW treatment capacity. In a period of five months from June to October 2019 a pilot green wall for GW treatment with a heat exchanger for GW heat recovery was designed and implemented in the lobby of the Faculty of Civil and Geodetic Engineering in Ljubljana (Slovenia). The selected design of the green wall in this study was a green wall with four cascading beds with horizontal flow. The wall height was 2.45 m tall. The frame of the pilot green wall was two stainless steel beams with handles, which supported four stainless steel cascaded beds (dimensions of each bed's length x height x width: 160 x 26 x 20 cm). A mix of perlite (2–6 mm), gravel (2–6 mm), and a small volume of coconut fibres in a ratio 1:1:0.02 was selected as a substrate for this experiment. The filter porosity was measured at 55%, though the green wall was designed based on 48% porosity as a precaution to prevent clogging. To ensure the denitrification process in the green wall, a 2 cm layer of coconut fibres, as a source of carbon, was added in the third bed. A synthetic GW mix was used in this experiment. The hydraulic retention time was 18 h. The wall operated under two modes: continuous flow and intermittent (batch) mode. Under continuous flow the inflow was set to 5.56 L/h, and in the in the batch mode the flow was changed to an intermittent flow of 11 L/h.

After the GW was prepared in the mixing tank (1000L) it was pumped into the heat exchanger (70 L) and then to the reservoir (200 L), from where it was pumped into the uppermost green wall's bed. The GW percolated horizontally through the four cascading beds and exited into the last collecting tank and from there, pumped into the sewage. Therefore, the system was not designed for GW circulation, as it was important to have control over the input and output nutrient concentrations.

Over a relatively short period of time, i.e. 127 days, from October 2019 to March 2020, a pilot-scale experiment was conducted for the purpose of establishing suitable biological wastewater treatment conditions and to furthermore provide recommendations and determine the design parameters for greywater treating green wall systems. The main chemical parameters regularly observed were COD, BOD, NH_4-N, NO_3-N, and PO_4-P. The main measured physical parameters were: GW flow entering the first bed, the temperature, oxygen, pH, redox, and the electrical conductivity of GW in the mixing tank, reservoir, and green wall beds.

The experiment was divided in three phases that describe three different setups. In I. Phase, the greywater used was heavily loaded, no aeration was used and no vegetation was planted. In II. Phase the greywater load was halved and aeration was introduced, and in III. Phase vegetation was planted. Throughout the experiment chemical and physical parameters were monitored. After the experiment was over, further improvements on the design were proposed.

The treatment efficiency of the pilot green wall at the end of experiment in III. Phase was higher than at the start. The findings in this study are based on a relatively small number of samples, all of which may be classified under the start-up period for the treatment system.

The green wall's COD removal efficiency reached 70% in the last phase. In addition, BOD removal efficiency reached 74% in III. Phase. It can be concluded that the green wall itself was capable to treat in average up to 206 mg/L of COD and up to 44 mg/L of BOD.

The average efficiency of ammonium treatment in the green wall reached 20% in III. Phase. Furthermore, in the beds the root zone was actively ventilated and consequently nitrification was expected, but not denitrification. Contrary to what was expected, the average reached efficiency for nitrate treatment in the green wall reached 72%. Phosphorous was not planned to be treated in the pilot green wall, wherefore no phosphorous treatment was achieved in any of the three phases.

Furthermore, the initial OLR of 68 g COD/m^2d and 16 g BOD/m^2d was too intense, and halving this load to 33 g COD/m^2d and 7 g BOD/m^2d might have been the bare maximum a system could sufficiently treat (with aeration, including the treatment in reservoir). To further improve overall treatment, the initial organic load can be decreased, when working with synthetic GW. However, controlling initial loads is not feasible when working with real GW. Therefore, a system would need to run for a longer period than 4 months. Longer hydraulic retention times than 18 hours should also be considered as well as longer (than 15 min) rest periods in a batch mode to promote aeration. Other studies that achieved better greywater treatment included much lower organic load and were carried out over a stretch of 8 to 22 months. The hydraulic retention time was also longer, i.e. from one day up to three days. Intermittent feeding happened once every hour.

Clearly the nitrification process did not reach a steady state under the period of study. This is in line with other findings that nitrification can take on the order of 1–3 months for nitrifying bacteria to fully develop in intermittent media filters (Bahgat *et al.* 1999). According to the measurements, it can be stated that nitrification was achieved to some extent; however, it was not as successful as was initially planned. This result is partly due to a number of changes that were made to the system in the limited time available for running the experiment, which basically proceeded under the long biological processes. Therefore, it is questionable whether enough time was available for the suitable microorganisms to grow. Nevertheless, focusing on treating nitrogen in substrate-plant based systems such as green walls is reasonable only when its concentrations are high, which with greywater usually is not the case.

The material used for substrate was lacking minerals such as Ca, Fe, Mg or Al for removal of phosphorous, thus substrate such as limestone rather than silica sand is recommended if dephosphatisation is needed.

Related to the oxygen concentration, it should be emphasised that it was hard to determine what kind of conditions were established along the green wall bed section. Therefore, it is proposed to take samples from the lower middle of the beds as well. Also, investigations of oxygen conditions using dissolved oxygen measurements and gas tracer studies along the substrate section would be valuable in order to characterize the effect of the green wall design.

Nevertheless, the greywater treatment system was shown to significantly reduce nutrient concentrations in the effluent water. In this regard, past the treatment, the effluent from the green wall design satisfied the existing guidelines for water reuse of most countries that already have them. Some effluent concentration satisfied US's standards, which for example require <10 mg BOD/L for unrestricted urban reuse (non-potable applications with uncontrolled public access). Certainly, the effluent concentrations satisfied the US's guidelines for environmental reuse requiring <30 mg/L in terms of BOD and for certain reuses under Italy's standards in terms of COD requirement requiring <100 mg/L. In regard to US's standards on turbidity, the results (8.7 NTU) were close to meeting the guideline for unrestricted urban reuse (<2 NTU) and agricultural reuse (<5 NTU).

Overall, the study shows that GW, which is a promising source of water for secondary use, nutrients, and thermal energy for preheating cold sanitary water, can be simultaneously employed in one continuous system. Also, the results of this study demonstrate that innovative and aesthetically pleasing vegetated green walls can be designed for treating GW and are a promising technology not only for outdoor but also for indoor, on-site, urban GW treatment at a household scale.

Although the green wall concept was not entirely verified, the end treatment results showed great potential for implementation. Further work can focus on heat recovery from greywater; microbial activity, the types of microorganisms both present in greywater and accountable for its biological treatment, optimizing plant selection with species that have a higher nutrient uptake, good acclimatisation to wetlands, and high levels of adaptation and survival in greywater. Other issues include the optimisation of a substrate selection that is both minerally and organically rich with grain sizes that facilitate phosphorous removal, denitrification, improved hydraulic conductivity through the removal of organic matter and suspended solids to reduce clogging, the optimisation of oxygen conditions by combining horizontal flow with vertical and by optimizing the intermittent feedings. These areas show a great opportunity for collaboration between wastewater treatment engineers, mechanical engineers, botanists, microbiologists, and biochemists.

12 POVZETEK

Hitra urbanizacija v kombinaciji s podnebnimi spremembami v zadnjih desetletjih negativno vpliva na naravne ekosisteme kot posledica nenehnega povečevanja potrebe po vodi in energiji (UNESCO, 2015, Strungaru *et al.,* 2015). Vsak dan je na Zemlji na voljo manj čiste pitne vode. Potreba po pitni vodi se bo na globalni ravni med letoma 2000 in 2050 povečala za 55% (OECD, 2012), medtem ko naj bi se potreba po energiji do leta 2035 letno povečevala za 1.3%, največ pa v Indiji že danes za 2.7% (International Energy Outlook, 2017). Posledično se povečuje povpraševanje tudi po novih in alternativnih vodnih in energetskih virih, kot sta na primer pridobivanje vode in prenos toplote iz odpadnih voda (Garnier *et al.,* 2015; Frijns *et al.*, 2013; Ravichandran *et al.*, 20202). Prenosnik toplote in toplotne črpalke (ali kombinacija obeh) se lahko uporabljata za pridobivanje toplote iz odpadne vode (Arnell *et al.,* 2017) in posledično prihranita energijo za ogrevanje tople gospodinjske vode, ki predstavlja 4-6% celotne porabe nacionalne energije v razvitih državah.

To magistrsko delo je osredotočeno zlasti na ločevanje sive vode (SV) od odpadne vode, njeno čiščenje in ponovno uporabo za sekundarne namene ter prenos toplote iz SV. Na splošno je SV označena kot rahlo onesnažena gospodinjska odpadna voda v industrializiranih državah, ki odteka iz pomivalnih strojev, tušev, umivalnikov, pomivalnih in pralnih strojev, z izjemo odpadne vode iz stranišč (Department of Health Western Australia, 2010; Environment Agency, 2011; Eriksson *et al.*, 2002; World Health Organization 2006; Friedler and Hadari 2006). Očiščeno SV je mogoče ponovno uporabiti v zaprtih prostorih za splakovanje stranišč ali pranje oblačil ter na prostem npr. za namakanje travnatih površin v kampusu, atletskih igriščih, pokopališčih, parkih, igriščih za golf, domačem vrtu (Okun, 1997), pranje vozil in oken, požarno zaščito, napajalno vodo kotla in proizvodnjo betona (Okun, 1997; Santala *et al.,* 1998). Prednosti ponovne uporabe SV so številne. Na primer, s ponovno uporabo SV se znižajo skupni stroški ravnanja z odpadno vodo, saj se količina odpadne vode, ki se izteka na čistilno napravo, zmanjša (Eriksson *et al.,* 2002). SV se lahko uporablja tako v turističnih krajih v hotelih kot tudi v drugih gosto naseljenih krajih z visokimi zgradbami, kjer sta potreba po vodi in proizvodnja SV največja. Hitra urbanizacija in eksponentna rast svetovnega prebivalstva vplivata na pojav sprememb v urbanem okolju na način gradnje visokih in "sivih" namesto "zelenih" mest. Nadomestitev vegetacije in naravnih površin z nenaravnim antropogenim okoljem, zgrajenim iz betona, opeke in asfalta, je vplivalo na spremembe v dinamiki urbanih ekosistemov (Shooshtarian *et al.,* 2018). Tovrstne spremembe v urbanem okolju in uporaba neprepustnih materialov lahko privedejo do zajemanja sončnega sevanja in poslabšanja prezračevanja v mestih. Posledično se pojavijo višje temperature, ki jih prepoznamo kot učinek mestnega toplotnega otoka (Palme *et al.,* 2016). Zato postajajo tehnologije, ki lahko odpadno vodo očistijo in ponovno uporabijo kot alternativni vir vode in energije s pridruženim hladilnim učinkom na zgradbe, ter upravljanje z vodami, zelo pomembni pri premagovanju omenjenih okoljskih problemov v urbanem okolju (Ghaitidak in Yadav, 2013; Li *et al.,* 2009; WWDR , 2015).

Obstaja ideja o tehnologiji, podobni rastlinskim čistilnim napravam, o morebitnem alternativnem sistemu za čiščenje odpadnih voda, poznana pod različnimi izrazi, kot so zelene stene, živi zidovi, mokre stene, mokre fasade itd. (Medl, Stangl in Florineth, 2017). Zelene stene lahko zagotavljajo zvočno udobje, ustvarjajo ekološke habitate z biotsko raznovrstnostjo, prevzamejo funkcijo izolacije stavb, čistijo zrak ter se uporabljajo za čiščenje odpadne vode. Slednje je glavna vsebina tega magistrskega dela, v katerem smo zbrali in določili dizajn parametre tovrstnega sistema. Prav tako smo na podlagi pregleda strokovne literature predlagali najprimernejši sistem za prenos toplote v kombinaciji z zelenimi stenami z namenom nadaljnjih študij o SV. Centraliziran sistem za prenos toplote iz SV s potopljeno spiralno navito cevjo, v katerem se topla SV zbira in teče skozi rezervoar in ogreva čisto vodo, ki teče v spiralno naviti cevi potopljeni v rezervoarju, smo prepoznali kot najprimernejši tip prenosnika toplote v kombinaciji z zeleno steno za čiščenje SV.

V času izdelave magistrskega dela smo zasnovali pilotno zeleno steno in opravili eksperimentalno delo z namenom zbiranja podatkov o njeni učinkovitosti čiščenja SV. Eksperimentalnega dela na področju prenosa toplote iz SV nismo opravili, saj bi presegalo obseg magistrskega dela. V obdobju petih mesecev od junija do oktobra 2019 smo v avli na Fakulteti za gradbeništvo in geodezijo, Hajdrihovi ulici 28, v Ljubljani, sestavili pilotno zeleno steno s prenosnimkom toplote. Za zasnovo zelene stene v tej študiji smo izbrali linearno zeleno steno s štirimi kaskadnimi koriti in vodoravnim tokom. Višina okvirja zelene stene znaša 2.45 m. Okvir pilotne zelene stene je bil izdelan po naročilu iz dveh nerjavečih jeklenih nosilcev z ročaji, ki podpirata štiri kaskadna jeklena korita (dimenzije: dolžina x višina x širina: 160 x 26 x 20 cm). Za substrat smo izbrali mešanico perlita (2–6 mm), gramoza (2–6 mm) in majhno količino kokosovih vlaken v razmerju 1:1:0.02. Izmerjena poroznost filtra je bila 55%. Kljub temu smo zeleno steno zasnovali na podlagi 48% poroznosti, kar je služilo kot previdnostni ukrep pri preprečevanju zamašitve. Z namenom zagotovitve procesa denitrifikacije, smo za vir ogljika v tretjo korito dodali 2 cm debelo plast kokosovih vlaken. Za odpadno vodo smo uporabili sintetično mešanico SV. Zadrževalni čas je bil 18 ur. Pretok smo nastavili na 5.56 L/h. Zaradi zamenjave iz kontinuirnega v šaržni sistem je bil pretok pozneje spremenjen na 11 L/h.

Po pripravi SV v cisterni (1000 L) je bila ta z računalniško vodenim sistemom črpalk prečrpana v prenosnik toplote (70 L) in nato v rezervoar (200 L), od koder je bila prečrpana v skrajno zgornje korito zelene stene. SV se je nato vodoravno prefiltrirala skozi štiri kaskadna korita in izstopila v zadnji zbiralnik od koder se je prečrpala v kanalizacijo. Sistem nismo zasnovali z namenom kroženja SV, saj nam je bilo pomembno imeti nadzor nad vhodnimi in izhodnimi koncentracijami hranil in organske obremenitve.

V sorazmerno kratkem času, tj. 127 dneh, od 30. oktobra 2019 do 4. marca 2020, smo izvedli laboratorijsko-pilotski poskus z namenom določitve primernih bioloških pogojev za čiščenje SV ter določitve dizajn parametrov za zelene stene s funkcijo čiščenja SV. Glavni redno opazovani kemijski parametri so bili KPK, BPK, NH_4-N, NO_3-N in PO_4-P. Glavni izmerjeni fizikalni parametri so bili: pretok SV, ki vstopa v prvo korito, temperatura, kisik, pH, redoks in električna prevodnost SV v cisterni, rezervoarju in koritih zelene stene.

Poskus smo razdelili v tri faze, ki opisujejo tri različne pristope. V I. fazi smo uporabili organsko močnejše obremenjeno SV, brez dodatnega vpihovanja zraka in zasaditve vegetacije v substratu. V II. fazi smo obremenitev SV prepolovili, v rezervoarju in koritih smo uvedli vpihovanje zraka ter v III. fazi posadili rastline. Tekom poskusa smo spremljali kemijske in fizikalne parametre. Po končanem poskusu smo podali predloge za izboljšanje pilotnega sistema.

Učinkovitost pilotne zelene stene ob koncu poskusa v III. fazi je bila višja kot na začetku. Zaradi kratkega eksperimentalnega obdobja zelena stena ni dosegla svoje potencialne učinkovitosti čiščenja. Ugotovitve v tej študiji temeljijo na sorazmerno majhnem številu vzorcev, ki jih na podlagi primerljivih znanstvenih študij lahko štejemo v obdobje zagona zelene stene.

Učinkovitost čiščenja zelene stene je za parameter KPK v zadnji III. fazi dosegla 70%. Poleg tega je učinkovitost odstranjevanja BPK v III. fazi dosegla 74%. Sklepamo lahko, da je bila zelena stena v povprečju sposobna očistiti do 206 mg KPK/L in 44 mg BPK/L.

Povprečna učinkovitost odstranjevanja amonija v zeleni steni je v III. fazi dosegla 20%. Poleg tega je bilo v koritih območje korenin aktivno prezračevano in je bila zato nitrifikacija pričakovana, ne pa tudi denitrifikacija. V nasprotju s pričakovanji je povprečna učinkovitost odstranjevanja nitratov v zeleni steni dosegla 72%. Odstranjevanje fosforja v pilotni zeleni steni ni bilo predvideno, zato v nobeni od treh faz odstranitve nismo zaznali.

Ugotovili smo, da je bila začetna organska stopnja obremenitve (OLR) 68 g KPK/m^2d in 16 g BPK/m^2d preintenzivna in, da bi bila lahko prepolovitev te obremenitve na 33 g KPK/m^2d in 7 g BPK/m^2d na celotno površino korit obremenitev, ki bi jo bil sistem (z vpihovanjem zraka), brez intenzivnega vonja, v obdobju zagona sposoben očistiti. Učinkovitosti čiščenja zelenih sten lahko pri uporabi sintetične SV izboljšamo z znižanjem začetne organske obremenitve. Vendar obvladovanje začetnih obremenitev pri uporabi prave SV ni izvedljivo. Zato predlagamo daljši obratovalni čas sistema kot 4 mesece, daljši hidravlični zadrževalni čas (HRT) od 18 ur in daljši čas premora med doziranjem SV z namenom spodbujanja prezračitve substrata od 15 minut. V primerjavi z drugimi študijami, ki so dosegle boljšo učinkovitost čiščenja SV, z običajno precej nižjo organsko obremenitvijo, so bili tovrstni poskusi opravljeni v obdobju od 8 do 22 mesecev, HRT je trajal od enega do tri dni, občasno doziranje SV pa je bilo nastavljeno enkrat na uro.

Jasno je, da se postopek nitrifikacije v obdobju študije ni stabiliziral. To je v skladu z ugotovitvami primerljivih študij, saj lahko nitrifikacija traja od 1 do 3 mesece, da se nitrificirajoče bakterije popolnoma razvijejo (Bahgat in sod. 1999). Nizka odstranitev amonijaka je v I. fazi predvsem posledica prevladujočih anoksičnih razmer v substratu, amonifikacije organskega dušika in temu posledično zaviranje nitrifikacije. Glede na meritve lahko sicer trdimo, da je bila nitrifikacija do neke mere dosežena, vendar manj uspešna, kot je bilo sprva načrtovano. Ta rezultat je deloma posledica številnih sprememb, ki smo jih uvedli v razpoložljivo kratkem času izvajanja poskusa, ki praktično temelji na dolgih bioloških procesih. Zato je vprašljivo, ali je bilo na voljo dovolj časa za rast ustreznih mikroorganizmov tudi v zadnji prevladujoče aerobni fazi. Kljub temu je osredotočanje na odstranjevanje dušika v sistemih, ki temeljijo na rastlinah, kot so zelene stene, smiselno le, če so njegove koncentracije visoke, kar pri SV običajno ni tako.

Pri vsebnosti izbranega materiala za substrat smo ugotovili, primanjkljaj mineralov kot so Ca, Fe, Mg ali Al, ki so potrebni pri odstranjevanju fosforja. Zato pri nadaljnjem raziskovanju priporočamo substrat kot je apnenec in ne kremenčev pesek, ki je bil uporabljen v tej študiji. Pri tem je potrebno upoštevati menjavo substrata na določeno obdobje, saj se s časom sposobnost adsorpcije fosforja zmanjša.

V saturirani coni substrata smo imeli težave določiti vzpostavljene pogoje koncentracije kisika v SV vzdolž korita. V nadaljnje predlagamo odvzem vzorcev tudi z dna sredine korit in vpeljavo dodatnih merilnih sistemov za spremljanje kisika vzdolž posameznega korita.

Proti koncu poskusa se je izkazalo, da je pilotni sistem za čiščenje SV sposoben znatno zmanjšati koncentracijo hranil in organsko onesnaženje v SV. V zvezi s tem so koncentracije snovi v SV po čiščenju zadovoljile obstoječe smernice za ponovno uporabo glede na BPK v večini držav, ki jih že imajo. Očiščeno SV v zeleni steni bi lahko na primer ponovno uporabili po ameriških standardih, ki zahtevajo <10 mg BPK/L, za neomejeno ponovno uporabo v mestih (z nenadzorovanim javnim dostopom) in za ponovno uporabo v okolju (z nadzorovanim javnim dostopom), kjer zahtevajo <30 mg BPK/L, ter v skladu z italijanskimi standardi, ki zahtevajo <100 mg KPK/L. Kar zadeva priporočila motnosti, so bili rezultati (8 NTU) blizu izpolnitve smernic za neomejeno ponovno uporabo v ZDA (<2 NTU) in kmetijsko rabo (<5 NTU).

V magistrskem delu smo dokazali, da je lahko SV obetaven vir za večkratno sekundarno uporabo vode, hranil in toplotne energije za ogrevanje hladne sanitarne vode, združeno v enem tehnološkem sistemu. Rezultati te študije dokazujejo, da so inovativne in estetsko prijetne zelene stene lahko zasnovane za čiščenje SV in so obetavna tehnologija ne samo za zunanjo, temveč tudi za notranjo uporabo.

Čeprav koncept zelene stene nismo popolnoma preverili, so končni rezultati čiščenja pokazali velik potencial za izboljšanje in nadgradnjo pilotnega sistema. Predlagamo, da se nadaljnje delo osredotoči na: prenos toplote iz sive vode; preiskovanje mikrobne aktivnosti in vrst mikroorganizmov prisotnih v

SV in odgovornih za njeno biološko čiščenje; optimizacijo izbire vegetacije osredotočeno na rastline z večjim vnosom hranil, dobro aklimatizacijo na mokriščih, prilagajanjem in preživetjem v SV; optimizacijo izbire substrata, ki vsebuje mineralne in organske komponente, izbiro velikosti zrn substrata, z namenom izboljšanja procesa denitrifikacije, odstranjevanja fosforja, organskih in suspendiranih snovi ter izboljšanja hidravlične prevodnosti za preprečevanje mašenja; optimizacijo koncentracije kisika v SV s kombiniranjem vodoravnega in navpičnega toka in z optimizacijo dovajanja SV v zeleno steno s prekinitvami. Vsi ti predlogi kažejo na odlično priložnost za sodelovanje med interdisciplinarnimi profili: okoljskimi inženirji, strojnimi inženirji, botaniki, mikrobiologi in biokemiki ter nova poglavja v znanosti.

13 LITERATURE

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