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The efficacy and sustainability of constructed wetlands in treating wastewater arising in an urban informal settlement

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A thesis submitted to the Faculty of Science, University of Witwatersrand, in the fulfilment of the requirements for the degree of Master of Science

Johannesburg, 2022

Declaration

I declare that this thesis is my own unaided work. It is being submitted for the degree of Master of Science to the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination to any other University.

T Rawhani

Taraz Rawhani

04 November 2022

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To the residents of Silvertown, Alexandra, who welcomed me with a warm embrace.

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List of Abbreviations

Abbreviation	Definition
BOD	Biological Oxygen Demand
CFU	Colony Forming Units
COD	Chemical Oxygen Demand
CW	Constructed Wetland
DRP	Dissolved Reactive Phosphorus
EC	Electrical Conductivity
EPA	Environmental Protection Agency
FWS	Free Water Surface
HF	Horizontal Flow
HRT	Hydraulic Retention Time
MPN	Most Probable Number
ORP	Oxidation Reduction Potential
PE	Population Equivalent
TC	Total Coliforms
TDS	Total Dissolved Solids
VF	Vertical Flow
VSB	Vegetated Submerged Bed
WERF	Water Environment Research Foundation
WWTP	Wastewater Treatment Plant

1 Introduction

1.1 Background

It is no secret that the world is currently facing, and will continue to face, challenges with water supply and pollution of water resources. The 2030 agenda for sustainable development, outlined by the United Nations, clearly highlights clean water and sanitation as one of the priority 17 goals towards which the world must work (THE 17 GOALS | Sustainable Development, n.d.). Furthermore, a report on South Africa's water challenges by McKinsey & Company predicts that by the year 2030, if we continue on our current trajectory, parts of South Africa will face a deficit between demand and supply of water between 20% to 80%- indicating that the 'Day Zero' only nearly averted in Cape Town may become a more common occurrence (Confronting South Africa's Water Challenge | McKinsey, n.d.). Providing clean water and sanitation in a country with a high rate of inequality and a vast difference in living standards between neighbouring areas, requires a multi-pronged and interdisciplinary approach.

In a 2014 paper delineating key areas of water research in South Africa, the Water Research Commission identified the hygienic and economical treatment of greywater arising in low-income and informal settlements as one of the research priorities for water in the country.

Treating greywater is important in the context of South Africa's informal spaces for two reasons. First, South Africa is increasingly becoming a water-scarce country, and the potential for greywater to become treated so that it may be recycled is significant in this regard (Carden et al., 2007). Second, Carden and colleagues find that while the exact composition of greywater varies from household to household, there is no doubt that where it is not treated effectively it can pose a threat to human health and well-being, and the environment, as it comes to contain pathogens capable of causing and spreading diseases of the skin, eyes, and gastric system; viruses; parasites which cause diseases like bilharzia; and bacteria which cause diseases like cholera (Carden et al., 2007). Groups particularly at risk of being impacted by such pathogens include people under the age of 19, especially children who are potentially doubly exposed not only through accidental consumption of such water but also due to carrying out recreational activities in areas where they come into contact with greywater; and immunocompromised adults such as those suffering from chronic diseases like HIV/AIDS, tuberculosis, the elderly, or pregnant women. In the informal context, where a high percentage of residents suffer from compromised health and

comparatively limited access to water, addressing these two negative corollaries of untreated greywater is of great importance (Carden et al., 2007).

Although there are many technologies such as the activated sludge process, membrane bioreactors and membrane separation (which are typically used in larger urban areas) that can treat greywater, many of them are relatively expensive and not feasible for widespread application in informal settlements (Chen et al., 2014)

One option that has emerged in the last few decades as a decentralized, low-cost, low operation and maintenance alternative to centralised wastewater treatment plants is the constructed wetland. Constructed wetlands have been widely applied greywater treatment operations for which centralised wastewater treatment facilities would not be feasible. Although many investigations into the technical design and optimisation of CWs have been conducted, few consider the involvement of the community in building, maintaining and correctly using CWs. This research will aim to investigate the efficacy of CWs that have been built by local community members, with locally sourced materials, and maintained with little external intervention in treating greywater arising in an informal settlement.

1.2 Hypotheses Under Study

The focus of this research project is to explore the efficacy of using constructed wetlands for the treatment of domestic effluent in an informal settlement in South Africa. The specific research hypotheses that are being tested in this study are shown in Table 1.

Table 1 List of Hypotheses that are being tested in this study

No.	Hypothesis
1	The constructed wetlands under study lower the concentration of coliform bacteria in domestic wastewater from Alexandra.
2	The constructed wetlands under study lower the concentration of chemical oxygen demand in domestic wastewater from Alexandra.
3	The constructed wetlands under study lower the concentrations of nutrient ions in domestic wastewater from Alexandra.
4	The constructed wetlands under study improve in-situ parameters of domestic wastewater from Alexandra. For pH, the improvement means the water becomes more circumneutral and for electrical conductivity improvement a decrease in the

	value. For oxidation-reduction potential the constructed wetlands will decrease the value.
5	Constructed wetlands can act as buffers of water quality, lowering the range of concentration/value of the pollutant/parameter.
6	The water quality of the Jukskei River deteriorates between London Road, Alexandra (upstream of Alexandra) and Frankenwald, Sandton (downstream of Alexandra).

1.3 Objectives of this Study

To test the hypotheses under study, this research will include the objectives listed in Table 2 which were approved by the University of Witwatersrand in the proposal that was submitted for this study.

Table 2 List of objectives that were approved in the research proposal that was submitted for this study

No.	**Objective**
1	To conduct an ongoing characterisation of water from the Jukskei River, both upstream and downstream of Alexandra, to better analyse the effect of anthropogenic pollution from the township on the river.
2	To design, construct and commission a set of constructed wetlands in Alexandra using, to the extent that is feasible, locally-procurable materials.
3	To analyse the effectiveness of the constructed wetland in treating domestic wastewater following successful construction. This data will include a chemical and biological analysis of influent and effluent from the constructed wetland.

1.4 Thesis Structure

This thesis is presented in 5 main chapters. Chapter 1 presents the introduction and thesis structure. Chapter 2 covers an expansive literature review of wastewater treatment and constructed wetlands. Chapter 3 present the methods and materials of this project including the study area, the design and construction of the constructed wetlands and the methods and materials for water characterization measurements. Chapter 4 presents the results obtained from the water quality measurements of the two constructed wetlands under study and the Jukskei River. Chapter 5 provides a detailed discussion of the results that were obtained as well as general recommendations and a conclusion.

2 Literature Review

2.1 Background

In 1977, the world witnessed the first intergovernmental conference dedicated exclusively to water: the Mar del Plata United Nations Conference held in Argentina which included delegates from 105 countries (World Water Council, n.d.). Since that time, increased attention has been given to issues of water supply and sanitation on the international stage with notable advancements being the United Nations (UN) declaration of the 'International Drinking Water Decade' from 1981-1990 and, in 2010, recognizing the right to access drinking water and sanitation as a basic human right on par with the right to food (World Water Council, n.d.). More recently, in 2015, the UN has reaffirmed the importance of access to clean water and sanitation for all by setting it as one of the Sustainable Development Goals to be achieved by 2030 (Goal 6, n.d.).

2.2 Models of Wastewater Management Infrastructure

In a report by the Water Environment Research Foundation (WERF), three basic models of infrastructure for wastewater management were delineated (Wallace & Knight, 2006). These models; the regional sewer network, cluster systems and single home systems; are defined and distinguished by the population density of the geographical region that each serves, as shown in Table 3 (Wallace & Knight, 2006).

Table 3: Wastewater Infrastructure Models. Adapted from: (Wallace & Knight, 2006)

Population Density	**Infrastructure Type**
High	Regional sewer networks
Medium	Cluster systems
Low	Single-home systems

2.2.1 Regional Sewer Networks

Dating back to 7000 BC, the practice of constructing centralized sewer systems in urban areas is by no means a new phenomenon (Bertrand-Krajewski, 2003). These primitive methods largely acted as wastewater reticulation systems that would drain wastewater to the nearest river without any treatment, such as the Cloaca Maxima, or great sewer, of Rome that was built around 550-500 BC (Smith, 1875). More recently, as access to water distribution systems in urban areas increased towards the end of the 19th century, flush toilets gained popularity, and with it, the volume of wastewater being discharged by

households also increased (Wallace & Knight, 2006). This in turn lead to the formalization and further development of regional sewer networks that would not only carry sewage away from population centres, but also treat it before being released into the environment (Wallace & Knight, 2006). Although wastewater treatment was initially limited to gravity sewers, activated sludge or trickling filters, and surface water outfalls, policy implemented in the 1970's provided the impetus for secondary treatment plants that included biological treatment and other advanced systems (Wallace & Knight, 2006). Today it is generally accepted that although this 'Big Pipe' model can address the wastewater management requirements of certain populations, it is economically and logistically infeasible for others, with communities living in informal settlements and urban slums being prime examples (Wallace & Knight, 2006).

2.2.2 Cluster Systems

Designed to cater for smaller and less populated geographic regions, cluster systems generally service communities with between 2-200 homes (Wallace & Knight, 2006). The main distinction between these systems and regional sewer networks is that cluster systems collect, treat and dispose of wastewater at each population node, rather than collecting the wastewater for an entire city centrally and then treating and disposing of it (Wallace & Knight, 2006). Another key difference, facilitated by the lower volumes of wastewater that cluster systems need to treat, is that these systems employ subsurface dispersal for effluent water, compared to surface water discharge which is utilized by most regional sewer networks (Wallace & Knight, 2006). The benefits of cluster systems include lower capital intensity and collection costs, and less requirement for specialization and technical expertise for operation and maintenance (Wallace & Knight, 2006). One study, conducted in Austria, showed that 15% of the population, that live in small communities of less than 500 residents and lack access to regional sewer networks, could benefit from cluster systems as a cost-effective solution (Haberl et al., 1998).

2.2.3 Single Home Systems

Servicing geographical areas with the lowest population densities are single-home systems. In developed countries, most single home systems employ a septic tank, a technology developed in France in the 1860's (Kriessl, 2002). The proliferation of septic tank use is largely attributable to two main factors. Firstly, an increase in the average volume of domestic wastewater with the advent of flush toilets. This put strain on the existing leach pit systems which would clog easily with raw sewage and would result in the accumulation

of biomass, sometimes called a biomat, in the soil, adversely affecting ability of the soil to accept wastewater (U.S. EPA, 1980). Secondly, towards the end of the 19th century, greater consciousness about the harmful effects of open sewage and the relationship between disease and sanitation was emerging, which led to a preference of underground sewage treatment methods (Wallace & Knight, 2006).

2.3 Decentralized and small-scale wastewater treatment systems

2.3.1 Constructed Wetlands

Definition of Wetlands

Wetlands are often described as the transitional zones between land and bodies of water. Historically, there has been a lack of consensus on the definition of wetlands with some definitions being very broad, and others quite limited. While there is no universally adopted definition of these ecosystems, the U.S Fish and Wildlife Service classifies wetlands as having one or more of the following properties: (1) the land supports predominantly hydrophytes, at least periodically throughout the year, (2) the substrate is mostly composed of undrained hydric soils, and (3) the substrate is non-soil and at some point during the growing season each year is water-saturated or covered by shallow water (Wallace & Knight, 2006). The most widely adopted definition comes from the Ramsar Convention of 1971, held under the auspices of the United Nations, and defines wetlands as "areas of marsh, fen, peatland or water, whether natural or artificial, permanent or temporary, with water that is static or flowing, fresh, brackish or salt, including areas of marine water the depth of which at low tide does not exceed six metres" which "may incorporate riparian and coastal zones adjacent to the wetlands, and islands or bodies of marine water deeper than six metres at low tide lying within the wetlands" (Dalu & Wasserman, 2022).

At a very broad level, wetlands can be categorized into those which occur naturally and those which are man-made, known as constructed wetlands (CWs). Constructed wetlands differ from natural wetland in several ways including that constructed wetlands are generally hydraulically isolated from the surrounding environment except for influent, effluent and incoming rain. This is an important distinction, as this research focus primarily on CWs.

Water Treatment Mechanisms

Sometimes referred to as the 'Earth's kidneys', wetlands play a vital role in water filtration and purification. Through a variety of biological, physical and chemical mechanisms, wetlands can transform and reduce pollutant loads (Wallace & Knight 2006). These mechanisms of transformation include aerobic and anaerobic degradation of organic matter; sedimentation (discrete particle settling), aggregation (flocculation) and interception of suspended solids; mineralization (ammonification), volatilization, oxidation (both aerobic and anaerobic) and reduction (denitrification) of nitrogenous compounds; bioaccumulation of phosphorus; reduction and oxidation of sulfate; and filtration, sedimentation, adsorption, and natural die-off of pathogenic organisms (Wallace & Knight 2006).

History of Constructed Wetlands

Constructed wetlands emerged from research conducted by Seidel in Germany in 1952 in which she investigated using wetlands of *Scirpus lacustris* (bulrush) to treat wastewater (Gearheart, 1992). This research was extended in 1975 by Kickuth, again in Germany, with his monitoring of municipal wastewater that was discharged into a natural marsh of *Phragmites* (reeds) (Attwater, 1995).

In the 1970's, Bob and John Kadlec undertook studies funded by the National Science Foundation at the University of Florida and the University of Michigan exploring the treatment of wastewater using natural wetlands (Wallace & Knight, 2006). This was the foundation for future operational systems and pilot studies of CWs in the USA and Canada (Attwater, 1995).

Types of Constructed Wetlands

The Water Environment Research Foundation (WERF) in the USA categorizes CWs for small-scale water treatment applications into four types:

1. Free water surface (FWS) CWs which are similar to natural marshes in that they have areas of open water (Wallace & Knight 2006).

2. Vegetated submerged bed (VSB) CWs which are comprised of a gravel or soil bed planted with hydrophytic vegetation and in which the water flows horizontally from inlet to outlet below the bed-media surface (Wallace & Knight 2006).

3. Vertical flow (VF) CWs which distribute influent water across the surface of a gravel or sand bed planted with hydrophytic vegetation. The water percolates down through the bed and plant root zone (Wallace & Knight 2006).

4. Sludge dewatering beds (reed beds) dewater sewage sludge by using the evapotranspiration capacity of emergent hydrophytic plants (typically *Phragmites*) (Wallace & Knight 2006).

In a report released by WERF (2006), it was found that 93% of the small-scale CW systems that were identified in one study were VSB constructed wetlands (Wallace & Knight 2006). The preferential use of VSB constructed wetlands can be attributed to the lower health risk associated with them as a result of concealing the water surface beneath the level of gravel (Wallace & Knight 2006).

More recently, Wu et al. (2015) further developed the classification of CWs adding hybrid CWs as a category that includes multi-stage CWs and combinations of horizontal and vertical flow CWs. They further added enhanced CWs as a category that includes CWs with artificial aeration, baffled flow, hybrid towery, step feeding and circular flow corridors (Wu et al., 2015). This classification hierarchy is shown in Figure 1.

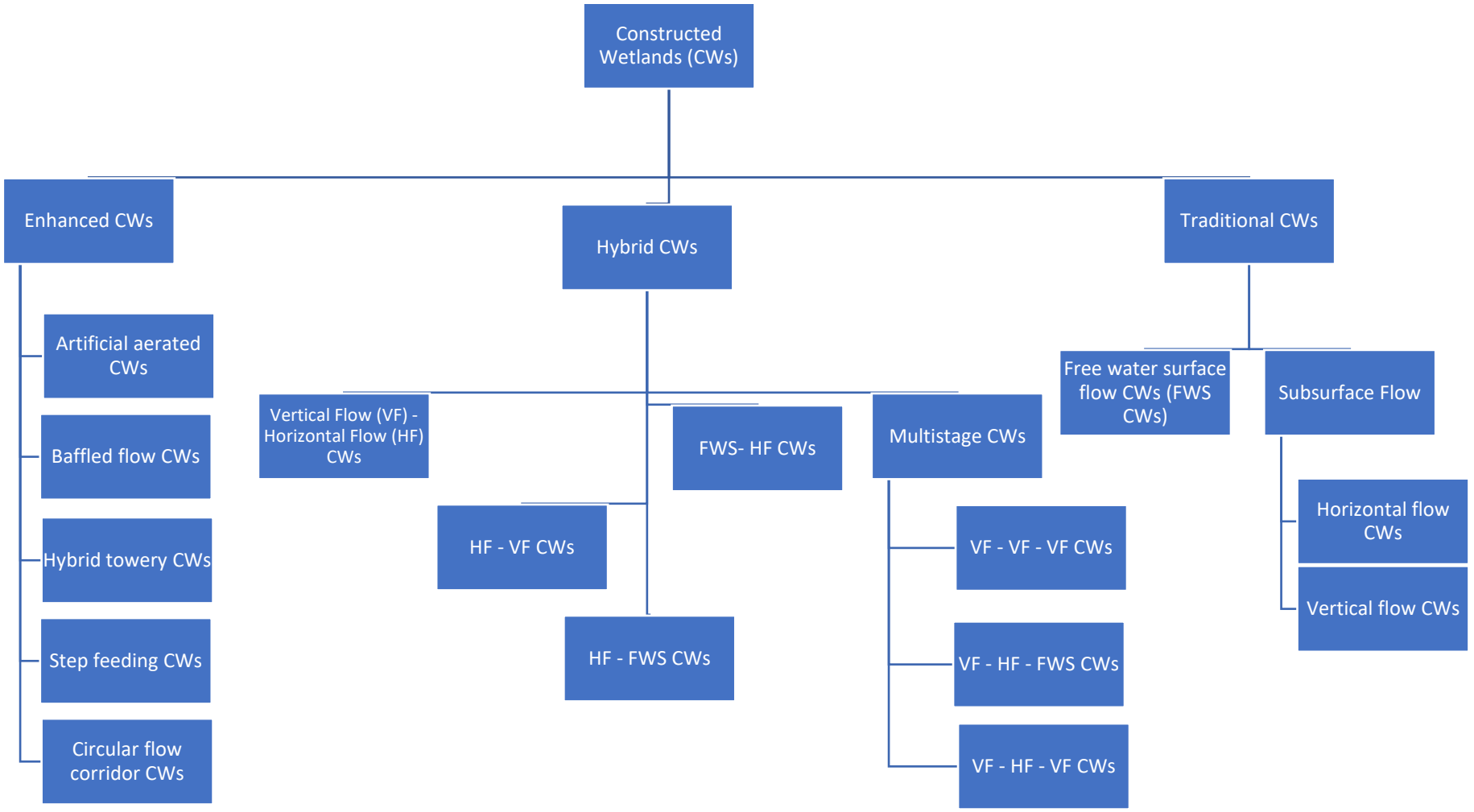

Figure 1: A classification of CWs used in wastewater treatment. Adapted from: Wu et al. (2015)

Some of the advantages and disadvantages of the use of CWs are shown in Table 4.

Table 4: Advantages and Disadvantages of Constructed Wetlands for Wastewater Treatment (Attwater, 1995)

Advantages	**Disadvantages**
Lower capital construction and operating costs than conventional systems	Require greater land base than conventional mechanical treatment systems
Consistent compliance with permit requirements	Reduced performance during vegetation establishment period
Operational and maintenance simplicity	Being seen as low maintenance system can often lead to no maintenance being conducted at all, which limits the effectiveness of the system.
Aesthetics may be greatly enhanced	For surface flow systems, mosquito control may be necessary
Habitat for wildlife	Potential for odour problems
High levels of wastewater treatment	Possible groundwater contamination
Can tolerate moderate fluctuations in water quality and flowrate	Standard design criteria are not readily available to engineers and regulators
	Lengthy start-up period
	Inability to remove nitrogen effectively without more complex design
	Potential for Sludge to accumulate and clog system
	Lack of adequate design protocol which can lead to underperformance of system
	It can be difficult to predict the performance of the constructed wetland based on the quality of incoming wastewater.

Constructed Wetland Design

Land

In general, CWs require more land for treatment per person than conventional wastewater treatment works or technologies such as trickling filters, activated sludge processes, up-flow anaerobic baffled reactors, moving bed biofilm reactors, and sequential batch reactors (Kivaisi, 2001; Parde et al., 2021). This is often one of the reasons that CWs are not adopted for wastewater treatment, especially in regions with high population density and expensive land (Zhang et al., 2014). There is a substantial range of values reported for CW land requirements in the literature. For example, Vymazal reported a land requirement of 5 m^2 PE^{-1} (PE= population equivalent = 60 BOD_5d^{-1}) for horizontal sub-surface flow CWs (Vymazal, 2010) while a report by the UN-HABITAT (UN-HABITAT, 2008), published only two years earlier, reported a 1-2 m^2 PE^{-1} requirement. This range of sizes can be attributed to the many variables involved in CW treatment systems including gravel size, ambient temperature, macrophyte choice, loading rates, retention times and wastewater composition.

Cost

A review of recent developments in the application of CWs for wastewater treatment by Ju et al. (2014) reported that CWs are more affordable for small communities than conventional wastewater treatment plants (WWTPs). A study by Liu et al. (2009) in China provided results confirming this statement, showing that the construction costs for CWs are one-third to one-half of those for WWTPs and the operation and maintenance costs are one-tenth to one-fifteenth. These costs can be reduced even further if energy-requiring equipment including pumps are not used. One parameter which is generally missing from the literature is the affordability of the construction costs to the local community. This is particularly important for the long-term maintenance and sustainability of CWs in developing countries (Massoud et al., 2009).

Hydrology

Hydraulic conditions are well-recognised to affect the treatment performance, biogeochemical processes and biotic community compositions of CWs (Kadlec & Wallace, 2008; Lee et al., 2009; Ranieri et al., 2013). Specifically, hydraulic retention time (HRT), the amount of time during which wastewater is in contact with the CW substrate and plant rhizospheres, is a vital independent variable that determines the CW's pollutant removal

efficiency (Stottmeister et al., 2003). One of the factors that greatly affects HRT is hydraulic loading rate, with a higher wastewater loading rate reducing optimum contact time with the CW substrate and therefore reducing pollutant removal efficiencies (Wu et al., 2015). Although a long HRT is generally desired for better CW removal efficiency, it requires more land and higher capital costs and is therefore not always feasible for projects in areas with high population density (Ju et al., 2014). Furthermore, not all pollutant removal efficiencies are equally affected by changes in HRT, as a study by Lee et al. (2009) showed. They reported that nitrogen removal, through the process of denitrification, requires a longer HRT than that required for the removal of organics (Lee et al., 2009). Optimal HRTs can vary between CWs as a result of variable parameters including pollutant concentrations, macrophyte choice and CW bed size (Kadlec & Wallace, 2008). A study by Ghosh and Gopal (2010) on the effect of HRT on the treatment of secondary effluent in a subsurface flow CW showed that optimum removal of NO3-N, total Kjeldahl nitrogen (TKN) and NH4-N was achieved with an HRT of 4 days. In a study of the effects of HRT on faecal coliform reduction, Khatiwada and Polprasert (1999) reported the figures shown in Table 5.

Table 5: Faecal Coliform Reduction for an Experimental Free Water Surface CW in Thailand. Adapted from: Khatiwada & Polprasert (1999)

Hydraulic Retention Time (HRT)	**Faecal Coliform Reduction**
1.5 days	84.26%
3 days	98.08%
5 days	98.09%
6 days	99.56%

Beyond HRT and hydraulic loading rate, water depth in CWs is one of the key hydrological factors that affects pollutant removal efficiencies. Water depth, largely decided by the bed depth of the CW, affects the dissolved oxygen levels and redox status of CWs, thereby influencing the biochemical reactions that break down pollutants in wastewater (Song et al., 2009). Historically, horizontal subsurface flow CWs in Europe were designed with bed depth of 0.6m (Wallace & Knight, 2006). This was due to a belief that the roots of Phragmites reeds planted in the CW would penetrate to this depth (Cooper & Boon, 1987). With further research however, it was found that macrophytes in CWs minimise their root penetration into the CW bed as a response to stresses from wastewater (Wallace & Knight,

2006). A study by Garcia et al. (2004) compared two horizontal subsurface flow CWs side by side, one with a bed depth of 0.27m and other 0.5m. The study reported that the CW with a shallower bed (0.27m) removed more ammonia, chemical oxygen demand (COD), five-day biochemical oxygen demand (BOD5) and dissolved reactive phosphorus (DRP) (García et al., 2004). This was attributed to the less-reducing conditions in the shallower CW, which also reportedly allowed for greater denitrification to occur (García et al., 2004).

Vegetation

Plants are an integral part of CWs and can include free-floating plants, emergent plants, floating leaved plants and submerged plants (Wallace & Knight, 2006; Wu et al., 2015). In general, it is reported that CWs with aquatic plants out-perform those without. This is attributed to the higher-density microbial communities that are found on and stimulated by macrophyte rhizospheres (Tanner, 2001; Vymazal & Kröpfelová, 2009). The presence of macrophytes, however, does not equally affect all types of CWs, and shows greater increase in pollutant removal efficiencies for free water surface CWs (Wallace & Knight, 2006). This is because the macrophyte rhizospheres provides the primary substrate for microbial communities in the absence of a bed media such as gravel (Wallace & Knight, 2006). In addition, macrophytes in CWs can also affect the redox status of wastewater under treatment as the plant roots can release oxygen which can stimulate the aerobic degradation of pollutants (Tanner et al., 2002). Building on these studies, Liu et al. (2013) showed that plants can accumulate toxic pollutants such as antibiotics and heavy metals, pollutants that are difficult to treat even in traditional wastewater treatment plants. Expanding this topic further, Yadav et al. (2012) found that the bioconcentration of heavy metals varied for different plant species, and above-ground plant biomass removed less metal than below-ground plant biomass. A further effect of the presence of macrophytes in CWs is the uptake of nutrients by plant growth, particularly in the warmer summer months, which increases pollutant removal efficiencies (Kantawanichkul et al., 2003; Kyambadde et al., 2004).

Although there are more than 150 species of macrophytes that are suitable for and have been used in CWs worldwide, in practice, only very few species are selected for use in the vast majority of CWs (Vymazal, 2013). These include *Juncus* spp. (Juncaceae), *Iris* spp. (Iridaceae), *Phragmites* spp. (Poaceae), *Scirpus* spp. (Cyperaceae), *Eleocharis* spp. (Spikerush) and *Typha* spp. (Typhaceae) (Wu et al., 2015).

Tolerance to wastewater is another important factor to consider in the selection of CW vegetation. The high levels of pollutants in wastewater can potentially exceed plant tolerance and reduce treatment efficiency and plant survivorship (Surrency, 1993). Extensive studies by Xu et al. (2010), for example, showed that high levels of COD in wastewater can have particularly adverse effects on *Phragmites australis*, the most frequently used plant in subsurface flow CWs (Vymazal, 2011). These effects include increased osmotic stress which affects plants growth, can induce stomatal closure and reduce cell expansion in root tips and young leaves; a reduced photosynthetic rate, chlorosis in leaves and both hypoxic and oxidative stress (Xu et al., 2010).

Wallace & Knight (2006) succinctly capture the most important factors that designers of CWs would need to consider in the selection of suitable plants. These are: hydrology, climate and latitude, cost and availability of planting stock, plant size, rate of colonization of the selected plant species, water quality (including salinity, pH, alkalinity and oxygen demand), project goals and maintenance requirements (Wallace & Knight, 2006).

Substrate

The substrate, or bed media, is one of the critical design parameters of CWs, especially in the design of subsurface-flow CWs (Kadlec & Wallace, 2008). The substrate ultimately provides the growing medium for wetland plants, and the hydrology of the CW is very much affected by differences in size and material of substrate (Wallace & Knight, 2006). Over the last two decades, much research has been conducted on the characteristics of different substrate materials, and there is a wide selection to choose from when designing a CW. Table 6 shows the various substrates that have been studied as bed media for CWs, which can largely be categorized into natural material, industrial by-product and artificial products (Wu et al., 2015). To aid in the selection of a suitable substrate, Wu et al. (2015) further identify two important characteristics to consider when selecting CW substrate: hydraulic permeability and pollutant sorption capacity.

Table 6: Substrates Commonly Selected for CW Wastewater Treatment. Adapted from: Wu et al. (2015)

Type of substrates	**Source**
Natural material	
Sand	(Saeed & Sun, 2013)
Gravel	(Calheiros et al., 2008)
Clay	(Calheiros et al., 2008)

Calcite	(Ann et al., 1999)
Marble	(Arias et al., 2001)
Vermiculite	(Arias et al., 2001)
Netonite	(Xu et al., 2006)
Dolomite	(Ann et al., 1999)
Limestone	(Tao & Wang, 2009)
Shell	(Seo et al., 2005)
Shale	(Saeed & Sun, 2012)
Peat	(Saeed & Sun, 2012)
Wollastonite	(Brooks et al., 2000)
Maerl	(Saeed & Sun, 2012)
Zeolite	(Bruch et al., 2011)
Industrial by-product	
Slag	(Cui et al., 2010)
Fly ash	(D. Xu et al., 2006)
Coal cinder	(Ren et al., 2007)
Alum sludge	(Babatunde et al., 2010)
Hollow brick crumbs	(Ren et al., 2007)
Moleanos limestone	(Mateus et al., 2012)
Wollastonite tailings	(Hill et al., 1997)
Oil palm shell	(Chong et al., 2013)
Artificial products	
Activated carbon	(Ren et al., 2007)
Light weight aggregates	(Saeed & Sun, 2012)
Compost	(Saeed & Sun, 2012)
Calcium silicate hydrate	(Li et al., 2011)
Ceramsite	(Li et al., 2011)

Historically, CW hydrology was designed based on the clean-bed hydraulic conductivity of the substrate (Reed et al., 1998; U.S. EPA, 1988). Later, however, it became evident that organic matter, debris and sediment accumulate over time in the CW which often results in clogging (Wallace & Knight, 2006). This prompted recommendations for clean bed

hydraulic conductivities to be reduced by a factor of 10 or even 100, so as to give a more accurate, 'real-world', figure (Kadlec et al., 1995; U.S. EPA, 2000; Wallace & Knight, 2006). The hydraulic conductivity of bed media can be estimated either by a set of formulae, such as the Beyer relationship equation, or from tabular heuristic data (Wallace & Knight, 2006). The U.S. EPA (2000) has outlined the 'clean' and 'dirty' hydraulic conductivities of different substrates, as shown in Table 7.

Table 7: Hydraulic Conductivities of "Clean" and "Dirty" Vegetated Submerged Bed Wetland Bed Media. Adapted from: U.S. EPA (2000)

Size and Type of Media	**"Clean"/"Dirty" Hydraulic Conductivity 'K' (m/d)**	**Type of Wastewater**	**Length of Operation**
5-10 mm gravel	34 000/12 000 (downstream section of bed)	Secondary effluent	2 years
5-10 mm gravel	34 000/900 (inlet section of bed)	Secondary effluent	2 years
17 mm creek rock	100 000/44 000	Nutrient solution	4 months
6mm pea gravel	21 000/9 000	Nutrient solution	4 months
30-40 mm coarse gravel	Not recorded/1 000 (inlet section of bed)	Secondary effluent	2 years
5-14 mm fine gravel	Not recorded/ 12 000 (downstream section of bed)	Secondary effluent	2 years
5 mm pea gravel	6 200/600	Landfill leachate	26 months
19 mm rock	120 000/3 000	Septic tank effluent	7 months

Beyond hydrological considerations for substrate selection, the sorption capacity of bed media is a vital parameter in the design of CWs. Chemically, substrates can remove contaminants from wastewater by precipitation, exchange, complexation and adsorption (Wu et al., 2015). These processes are particularly important for the removal of pollutants

that are not readily be broken down or absorbed by plants, for example: phosphorus (Ju et al., 2014). In fact, phosphorus is particularly important as it is often the limiting nutrient in fresh water systems and can adversely affect the flora and fauna in downstream receiving waters (Wallace & Knight, 2006). Although plant uptake is one of the mechanisms that removes phosphorus from wastewater in CWs, this process reaches equilibrium within the first few growing seasons and thereafter no longer significantly contributes to phosphorus removal from wastewater (Wallace & Knight, 2006). Similarly, if gravel is used as the bed media, adsorption sites are few and are quickly depleted (Wallace & Knight, 2006). For soil-based CWs, studies conducted in Germany have shown that the addition of aluminium and iron oxides to the substrate can increase phosphorus adsorption capacity (Kickuth, 1983; Rustige et al., 2003). Further north in the continent, Arias et al. (2001) studied 13 types of Danish sand and reported that the most important physico-chemical characteristic of sand in determining its phosphorus sorption capacity was its calcium content. The phosphorus removal capacity of sands with low calcium content was depleted after only a few months in the CW, whereas CWs with higher calcium content sand could persist for longer (Arias et al., 2001). Building on these studies, Xu et al. (2006) further showed the importance of substrate selection by comparing the phosphorus removal efficiencies of nine different materials: four types of sand, two types of soil, bentonite and two industrial by-products of furnace slag and fly ash. The results indicated that the worst performing substrate was one of the sands with a sorption capacity of 0.13 g P kg^{-1} and a lifetime of 9 months in a full-scale system, while the best performing substrate, furnace slag, was almost two orders of magnitude more effective with a sorption capacity of 8.89 g P kg^{-1} and a lifetime of up to 22 years (Xu et al., 2006). These results further emphasize the importance of substrate selection in the effectiveness and longevity of CWs in wastewater treatment.

Seasonal Variation and Water Temperature

The main mechanism of nutrient reduction in CWs is by biotic activity, a temperature-dependant process (Zhang et al., 2014). Consequently, the ambient temperature around the CW is of great importance in determining the pollutant removal efficiency of the system. As a rule of thumb, the pollutant removal efficiency of CWs decreases with lower temperatures as a result of the reduction of biotic activity (Zhang et al., 2014). In climatic zones that reach freezing temperatures, both subsurface and free water-surface CWs can develop a layer of ice (Wallace & Knight, 2006). Steps can be taken, however, to reduce

the likelihood of ice formation and the effects of ice formation on pollutant removal efficiency in CWs.

In subsurface CWs, a layer of straw, hay, leaf litter or other similar material covering the bed media can act as an insulating layer and prevent freezing if snow cover is not present (Kadlec, 2001; McCarthy et al., 1997). Furthermore, the part most susceptible to freezing in a subsurface CW is the outlet, and this is of particular importance as freezing of the outlet can block effluent water and cause ponding on the surface. To counteract this, material such as Styrofoam or hay bales can used to insulate the outlet, and in extreme conditions an electrically powered heat source such as an incandescent lightbulb can be used to maintain above-freezing temperatures (Wallace & Knight, 2006).

In free water surface CWs, the 'raise-and-freeze' method, whereby the water level is temporarily raised, an ice-layer is formed, and the water level is then lowered again to create an insulating air gap underneath the ice, can be used to provide a layer of insulation in cold conditions (Wallace & Knight, 2006).

Fortunately, in regions of Africa that seldom experience below-freezing temperatures, the above-mentioned precautions are not generally necessary, but are useful to maintain CW effectiveness during cold-fronts (Wallace & Knight, 2006).

Influent Feeding Method

Often overlooked, the feeding mode of the influent (such as batch, continuous and intermittent) to CWs is another important design parameter (Zhang et al., 2012). By altering the oxygen transfer and diffusion properties of the wetland, and influencing the oxidation-reduction conditions within it, different feeding modes can affect the overall treatment efficiency of the wetland (Wu et al., 2015). Although different types of CWs are affected in different ways by feeding mode selections, Wu et al. (2015) has reported that, in general, employing a batch feeding mode in CWs yields the best overall pollutant removal efficiencies. This can be attributed to the more oxidising conditions that batch feeding promotes (Wu et al., 2015).

A study by Zhang et al. (2012), compared the pollutant removal efficiency of continuous and batch feeding modes in a tropical subsurface flow CW. They showed that a batch feeding mode yielded significantly higher removal efficiency for ammonia (95.2% for batch compared to 80.4% for continuous) (Zhang et al., 2012). Furthermore, Saeed and Sun

(2012) showed that the removal of nitrogen and organics in CWs can be enhanced by adopting an intermittent feeding mode. Conversely, for sulfate removal in vertical flow CWs, Jia et al. (2010) reported that continuous feeding systems promoted higher removal than intermittent feeding systems. In conclusion: although batch feeding systems yield greater pollutant removal efficiency for subsurface flow CWs in general, other types of CWs can benefit from continuous feeding and this can be determined through testing and optimisation.

Pollutant Removal Mechanisms in Constructed Wetlands

Wetland treatment systems involve complex and intricate processes that transform and reduce pollutants (Wallace & Knight, 2006). These processes, involve biological, chemical and physical mechanisms including pathogen reduction, nutrient uptake by wetland vegetation, nitrogen transformations, organic matter degradation, sulfur cycling, phosphorus cycling and suspended solids removal (Wallace & Knight, 2006).

Pathogen Reduction

Often, a deciding factor in the selection of CWs for wastewater treatment is their ability to reduce the concentration of pathogens including helminths, bacteria, protozoa, viruses and fungi (Norton, 2007). The presence and concentration of *Escherichia coli* (*E. coli*) is often used as a measurement of pathogen contamination and an indicator of the presence of human faeces (World Health Organization, 2006).

FWS vs VSB Wetlands

Both free water surface and vegetated submerged bed CWs have been shown to be effective in the removal of pathogens. The dominant pathogen removal mechanisms are filtration, sedimentation and adsorption on sediments in free water surface CWS, and sedimentation, filtration and interception in vegetated submerged CWs, although relatively little research has been done on internal removal mechanisms in vegetated submerged bed CWs (Wallace & Knight, 2006). Pathogen removal in free water surface CWs also benefits from ultraviolet radiation in open waters, however the relative contribution of this mechanism towards overall pathogen reduction is not currently known (Mara, 1976). In both types of CWs, three days of retention time has been shown to yield a 98%-99% reduction in Faecal Coliform concentration (Gerba et al., 1999; Khatiwada & Polprasert, 1999; Thurston et al., 2001). In vegetated submerged bed CWs, these high removal efficiencies are associated

with incorporating smaller bed media, which yields a larger substrate surface area, and ensuring the CW is sized to approximately $3m^2$/population equivalent (García et al., 2003).

Nutrient Uptake by Wetland Vegetation

Of the pollutant removal mechanisms in CWs, nutrient uptake by macrophytes is one of the less studied mechanisms, and therefore there is less consensus among researchers regarding the overall importance of macrophytes in constructed wetlands. A study by Greenway (2003), showed that water in CWs typically contains a higher concentration of nutrients, and as a result, this leads to plant tissue from CWs containing higher nutrient contents. Furthermore, studies have been conducted measuring the nutrient uptake capacity of various types of macrophytes for various nutrients. For example, studies by Brix (2003) and Vymazal (1995) quantified the nutrient uptake capacity of emergent macrophytes: 30-150 kg Potassium/ha·yr and 200-2500 kg Nitrogen/ha·yr. These uptake figures are based on several assumptions, including the pivotal assumption that plant biomass is harvested at the correct time and season. If plant biomass is left to decompose in the wetland, the vast majority of the nutrients that were removed will be returned to the water column as they degrade (Wallace & Knight, 2006).

Nitrogen Transformations

Of vital importance in the treatment of wastewater, is the transformation and removal of nitrogen compounds (Norton, 2007; Wallace & Knight, 2006). Excess nitrogen in water can lead to many health and environmental problems including infant methemoglobinemia (‘blue baby syndrome’), caused by high concentrations of nitrate; toxicity to marine and aquatic life and eutrophication with its associated effects of dissolved oxygen depletion (Norton, 2007).

Nitrogen can exist in a variety of forms but is generally grouped into two categories: organic and inorganic nitrogen (Norton, 2007). Organic nitrogen includes sources such as urea, proteins, amino acids and nucleic acids (Prangnell et al., 2019). Common forms of inorganic nitrogen in wastewaters are ammonium, nitrate, nitrite and nitrogen or dinitrogen gas (Dodds & Whiles, 2020). The relative concentrations of the various compounds of nitrogen in wastewaters are dependent on the reduction/oxidation conditions of the water (Wallace & Knight, 2006). Wastewaters with higher concentrations of dissolved oxygen will contain greater amount of nitrates and nitrites, whilst wastewaters with reducing conditions will contain more ammonium (Norton, 2007).

There are numerous processes which lead to nitrogen transformation and removal in CWs. These include mineralization (ammonification), ammonia volatilization, proteobacterial ammonia oxidation, aerobic nitrite oxidation, anaerobic ammonium oxidation (annamox), nitrogen reduction (denitrification), organic nitrogen burial, ammonia adsorption, fixation and plant and microbial uptake (Norton, 2007; Wallace & Knight, 2006).

Mineralization (Ammonification)

In municipal wastewaters, about half of the overall nitrogen content, when received at wastewater treatment works, is in the form of organic nitrogen (U.S. EPA, 2000). The process whereby organic nitrogen is biologically transformed into ammonium nitrogen is referred to as mineralization, ammonification or hydrolysis (U.S. EPA, 2000). Although this process can occur under both anaerobic and aerobic conditions, studies have shown that the former results in a slower reaction rate (Mitsch & Gosselink, 2015). The rate of ammonification is largely dependent on the available nutrients in the system, pH, temperature and the C/N ratio of residue in the CW (U.S. EPA, 2000; Wallace & Knight, 2006). The optimum pH for this process has been reported at between 6.5 and 8.5 (Wallace & Knight, 2006). Studies have also shown that a temperature increase of 10º C results in a doubling of the rate of aerobic ammonification (Reddy et al., 1979). Mineralization of organic nitrogen is extremely important as it is generally regarded as a prerequisite to other nitrogen transformations (Wallace & Knight, 2006). After organic nitrogen has been hydrolysed to form ammonia it can then be volatilized in the form of gaseous ammonia, immobilised by ion exchange in the CW sediments, absorbed by macrophytes and hydrophytes through their rhizospheres, aerobically nitrified by nitrifying bacteria and even converted back into organic nitrogen by anaerobic microorganisms (U.S. EPA, 2000).

Ammonia Volatilization

Studies by Snoeyink and Jenkins (1980) confirmed that there is a chemical equilibrium between aqueous ammonia ($NH_{3(aq)}$) and the hydrolysed ammonium ion (NH_4^+), as shown in Equation 1.

Equation 1: Chemical equilibrium between aqueous ammonia and hydrolysed ammonia

$$NH_{3(aq)} + H_2O \rightleftharpoons NH_4^+ + OH^-$$

The ammonium ion is physiochemically in equilibrium between hydroxyl and gaseous forms, allowing some of the ammonium-nitrogen to be released to atmosphere, and this

process is called ammonia volatilization (Norton, 2007). Further research by Vymazal (1995), showed that the rate of this process is dependent on many factors including wind velocity, temperature, solar radiation, the concentration of NH_4^+ in water, the number and type of hydrophytes present and the system capacity to change pH during daily cycles. For example, the rate of ammonia volatilization is negligible if the system pH is below 8 (Reddy et al., 1984). However, at a higher pH of 9.3, the molar ratio of ammonia ($NH_{3(aq)}$) to ammonium (NH_4^+) is 1:1 leading to a more significant rate of ammonia volatilization (Wallace & Knight, 2006).

Proteobacterial Ammonia Oxidation

Ammonia can also be oxidised to form nitrite and nitrogen oxides by proteobacteria from the genera of *Nitrosococcus*, *Nitrosomonas* and *Nitrospira* (Wallace & Knight, 2006). This process was initially thought to only occur under aerobic conditions, but recent studies have discovered that these bacteria also have an alternate anaerobic metabolism (Wallace & Knight, 2006). The exergonic aerobic oxidation follows the reaction shown in Equation 2 (Wallace & Nicholas, 1969).

Equation 2: Exergonic aerobic ammonia oxidation reaction

$$NH_4^+ + 1.5O_2 \longrightarrow NO_2^- + 2H^+ + H_2O \quad [\Delta G^{o'} = -275 \frac{kJ}{mol}]$$

As shown in Equation 3 , the anaerobic oxidation pathway is more exergonic and therefore yields more energy, amounting to an additional -20kJ/mol (Schmidt et al., 2002, 2003).

Equation 3: Anaerobic oxidation pathway for ammonia

$$NH_4^+ + N_2O_4 \longrightarrow 0.33NO_2^- + 1.33H^+ + 0.22N_2 + 2NO + 1.33H_2O \quad [\Delta G^{o'} = -295 \frac{kJ}{mol}]$$

In both of the aerobic and anaerobic pathway, the same ammonia monooxygenase enzyme is utilised (Wallace & Knight, 2006). As can be seen in the reactions above, the anaerobic reaction utilizes dinitrogen tetroxide (N_2O_4) as an electron acceptor rather than nitrogen dioxide (NO_2), as in the aerobic reaction. Dinitrogen tetroxide is a dimer of nitrogen dioxide which is not produced in anaerobic conditions (Wallace & Knight, 2006). Therefore, the anaerobic reaction is dependent on there being an upstream aerobic area in which nitrogen dioxide can be formed and then converted to dinitrogen tetroxide (Wallace & Knight, 2006).

Aerobic Nitrite Oxidation

Aerobic nitrite oxidation is carried out by bacteria from the genera *Nitrospira* and *Nitrobacter* (*Schmidt, 1982* ; Wallace & Nicholas, 1969). This follows the general reaction shown in Equation 4.

Equation 4: Equation for aerobic nitrite oxidation

$$NO_2^- + 0.5O_2 \longrightarrow NO_3^- \quad [\Delta G^{o'} = -74\frac{kJ}{mol}]$$

As the name implies, this reaction requires aerobic conditions with a minimum dissolved oxygen concentration of 0.5 mg/L (Hanaki et al., 1990).

Anaerobic Ammonium Oxidation (Annamox)

Discovered in the early 1990's in wastewater sludge, anaerobic ammonium oxidation (annamox) bacteria are capable of using nitrite or nitrate to oxidise ammonium and form nitrogen gas (Kuenen, 2008). It was later found that these bacteria are responsible for up to 50% of the nitrogen consumption in marine environments (Kuenen, 2008). These bacteria belong the group Planctomycetales and include the species candidatus *Scalindua sorokinii,* candidatus *Kuenia stuttgartiensis* and candidatus *Brocardia anammoxidans* (Kuypers et al., 2003). The general chemical pathway for this oxidation reaction is shown in Equation 5 (Jetten et al., 1998, 2001; Strous et al., 1997).

Equation 5: General chemical pathway for anaerobic ammonium oxidation

$$NH_4^+ + 1.32NO_2^- + 1.066HCO_3^- + 0.13H^+ \longrightarrow 1.02N_2 + 0.26NO_3^- + 0.066CH_2O_{0.5}N_{0.15} + 2.03H_2O \quad [\Delta G^{o'} = -357kJmol]$$

Due to their anaerobic nature, anammox bacteria have been reported to be very sensitive to local oxygen concentrations and their growth and activity sharply declines with even low concentrations of oxygen (Jetten et al., 1998). The energy produced from their reaction pathway is significantly greater than the aerobic route (-357 kJ/mol vs. -257kJ/mol), however anammox's growth rate is far slower (doubling time of 10.6 days vs. 0.73 days) (Jetten et al., 2001; Wallace & Knight, 2006).

Nitrogen Reduction (Denitrification)

Denitrification, also referred to as nitrogen reduction or dissimilatory nitrate reduction, is the process whereby microorganisms reduce oxidised nitrogen compounds (e.g. nitrite or

nitrate) to gaseous forms of nitrogen (N_2O and N_2) which then vent and are expelled from the water column (U.S. EPA, 2000; Wallace & Knight, 2006). This process most effectively occurs under anaerobic and anoxic conditions, however it has also been reported to occur under aerobic conditions (U.S. EPA, 2000; Wallace & Knight, 2006). Denitrification proceeds according to the reaction pathway shown in Equation 6 and Equation 7.

Equation 6: Nitrate reduction pathway

$$4(CH_2O) + 4NO_3^- \rightarrow 4HCO_3^- + 2N_2O_{(g)} + 2H_2O$$

Equation 7: Nitrate Reduction Pathway

$$5(CH_2O) + 4NO_3^- \rightarrow H_2CO_3 + 4HCO_3^- + 2N_{2(g)}\, H_2O$$

Where (CH_2O) is the carbonaceous biodegradable matter required in the process.

As can be seen in the reaction equations, a source of carbon (organic matter) is required for the denitrification process to occur. This carbon is often derived from the CBOD of influent waters, chemical dosing (e.g. methanol) in the CW or the decaying matter of plant detritus (Wallace & Knight, 2006). A parameter that is often used to optimise this process is the carbon to nitrogen (C:N) ratio in the CW, with the optimal C:N ratio reported as 5:1 (Baker, 1998; Wallace & Knight, 2006).

Figure 2 shows the various nitrogen transformations that take place in a FWS CW (U.S. EPA, 2000)

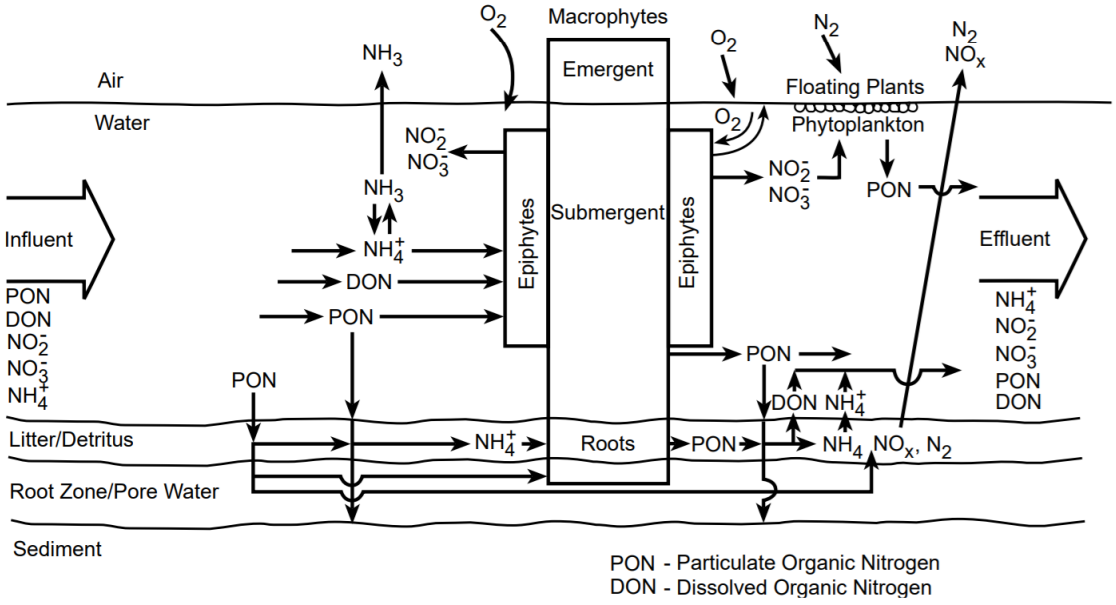

Figure 2 Nitrogen transformations in a free water surface constructed wetland (from: U.S. EPA, 2000)

Organic Matter Degradation

Organic matter in wastewater is composed of a wide variety of organic compounds and other substances that deplete dissolved oxygen in water (Wallace & Knight, 2006). These pollutants are present in both particulate (insoluble) and soluble forms, and range from being completely non-biodegradable to highly degradable by microbial action (Wallace & Knight, 2006).

Degradation of organic matter occurs largely via two types of decomposition: aerobic degradation and anaerobic degradation (Wallace & Knight, 2006). The prevailing conditions in the CW dictate which type of decomposition will be most prominent, and are largely determined by the oxygen transfer rate in the wetland and the magnitude of organic loading (Wallace & Knight, 2006). A CW with low organic loading in influent water and a high enough oxygen transfer rate to oxidise organic matter, will create aerobic conditions with the converse balance resulting in anaerobic conditions (Wallace & Knight, 2006).

Free water surface CWs are able to sustain both aerobic degradation conditions, in the upper layer of the water column where the water surface is in contact with air, and anaerobic degradation conditions, in the bottom layer of the water column (Wallace & Knight, 2006). In contrast, VSB CWS, are typically anaerobic systems, but still contain some dissolved oxygen in the upper layer of the water column (Wallace & Knight, 2006). Supporting this, a study by Garcia et al. (2004) showed that VSB CWs with a water depth of 0.27 m had higher removal efficiencies for COD, BOD, ammonia and dissolved reactive phosphorus compared to beds with a 0.5 m water column.

Soluble organic matter is degraded aerobically, or oxidised, by aerobic heterotrophic bacteria as shown in Equation 8 (Wallace & Knight, 2006).

Equation 8: Aerobic oxidation of organic matter

$$(CH_2O) + O_2 \rightarrow CO_2 + H_2$$

In contrast to the single step process of aerobic degradation, which is rapid and results in less accumulation of organic matter, anaerobic degradation of organic matter is a multi-step process (Cooper et al., 1996). It is initiated with a fermentation process which produces acetic acid and hydrogen gas as products, show in Equation 9 (Wallace & Knight, 2006).

Equation 9: Anaerobic fermentation step

$$C_6H_{12}O_6 \rightarrow 3CH_3COOH + H_{2(g)}$$

Following this fermentation step, anaerobic methanogenic bacteria convert acetic acid and hydrogen to methane and water shown in Equation 10 (Wallace & Knight, 2006).

Equation 10: Conversion of acetic acid and hydrogen gas to methane and water

$$3CH_3COOH + 4H_{2(g)} \rightarrow 2CH_4 + 2H_2O$$

The methanogenic bacteria in the 2nd step of the process are sensitive to pH fluctuations and will only operate between a pH of 6.5 to 7.5 (Wallace & Knight, 2006).

An important parameter in determining the type and rate of organic matter degradation, which can also be easily measured by handheld meters, is the oxidation-reduction potential (ORP) of water. Measured in mV, a positive ORP value is indicative of oxidative conditions, which is generally associated with aerobic conditions; and a negative value is indicative of reductive conditions, which is generally associated with anaerobic conditions (Myers, 2019; YSI, 2021). Yellow Springs Instruments (YSI, 2021) delineates various biochemical reactions that occur in wastewater and the corresponding ORP range associated with their occurrence, as shown in Table 8.

Table 8: Biochemical Reactions and Corresponding ORP Values. Adapted from: (YSI, 2021)

Biochemical Reaction	**Oxidation Reduction Potential (ORP), mV**
Nitrification	+100 to +350
cBOD degradation with free molecular oxygen	+50 to +250
Biological phosphorus removal	+25 to +250
Denitrification	+50 to -50
Sulfide (H_2S) formation	-50 to -250
Biological phosphorus release	-100 to -250
Acid formation (fermentation)	-100 to -225
Methane production	-175 to -400

Sulfur Cycling

In surface waters, sulfate (SO_4^{2-}) is generally the most abundant anion present (Wallace & Knight, 2006). However, due to its relatively low level of harm posed to humans and the environment, it does not form part of the discharge standards of wastewater to water resources (Department of Water and Environmental Affairs of South Africa, 2013). In

wetlands, the process of sulfide oxidation is indicative of aerobic conditions, whereas the reverse process of sulfate reduction indicates anaerobic conditions (Wallace & Knight, 2006). As a result of the oxidation-reduction potential of these sulfur compounds, sulfide creates an additional oxygen demand in wastewaters, while sulfate creates an additional oxygen supply (Wallace & Knight, 2006). In CWs treating wastewaters that contain high concentrations of heavy metals, e.g. mine drainage, the wetland system is intentionally designed to reduce sulfate to sulfide (Eger, 1992). This conversion is carried out by sulfate-reducing anaerobic bacteria as shown in Equation 11 (Wallace & Knight, 2006).

Equation 11: Reaction of sulfate to sulfide in constructed wetlands

$$SO_4^{2-} + organic\ compounds \rightarrow S_2^- + CO_{2(g)} + H_2O$$

Sulfide then reacts with heavy-metals in solution producing precipitate metal-sulfides that are highly insoluble and therefore easy to separate from the wastewater (Palmer et al., 1988). Whilst SO4 is not harmful, sulfide is very harmful, and this must be considered if these mechanisms are considered for sulfate removal.

Phosphorus Cycling

Removal of phosphorus from wastewater is an important element of treatment, particularly from wastewater that feeds to surface waters such as rivers and lakes. This is because phosphorus is generally the limiting nutrient in these fresh water ecosystems, and can thus lead to severe eutrophication in receiving waters (Wallace & Knight, 2006).

In FWS CWs, phosphorus is primarily removed from wastewater by sorption onto sediments, however, due to the relatively high concentration of phosphorus in wastewaters, and the low sorption threshold of wetlands sediments- this mechanism is quickly exhausted (Kadlec et al., 1995).

VSB CWs, on the other hand, have a greater capacity for phosphorus removal due to containing a bed media with many more sorption sites (Wallace & Knight, 2006). The bed media that is selected, however, can greatly influence the phosphorus removal capacity of the CW, but all bed media have a finite sorption capacity (Wallace & Knight, 2006). Gravel, a relatively common CW substrate, contains few sorption sites that are often quickly exhausted (Wallace & Knight, 2006). By contrast, Kickuth (1983) and Rustige (2003) reported that adding aluminium and iron oxides to soil-based CW systems could increase the phosphorus adsorption capacity. Man-made materials have been shown to have even

greater sorption capacity that natural materials. Jenssen et al. (2005) showed that, for horizontal flow VSB CWs that are preceded by a septic tank followed by an aerobic vertical down-flow biofilter, and which make use of light expanded aggregate clay (LECA) as a bed media to treat domestic effluent, phosphorus removal of > 90% can be expected for 15 years. In addition to sorption, phosphorus is also removed by plant uptake and sedimentation, although these processes are minor contributors (Wallace & Knight, 2006).

Suspended Solids Removal

CWs can be particularly well-suited to treating suspended solids in waste water (Wallace & Knight, 2006). The mechanisms that chiefly contribute to the removal of suspended solids include sedimentation, aggregation and interception (Wallace & Knight, 2006).

Sedimentation

Sedimentation, also referred to as discrete particle settling, is a natural process whereby particles settle in a fluid gravimetrically (Norton, 2007). The rate at which particles settle is related to the particle shape, specific gravity, size and the properties of the fluid (Norton, 2007). Particles with higher specific gravities and larger sizes will principally settle in the inlet area of the CW, with smaller particles requiring additional retention time and distance to settle (Wallace & Knight, 2006). The rate of particle settling can be calculated using Newton's or Stoke's Laws with the applicable drag coefficients (Kadlec et al., 1995; Norton, 2007).

Aggregation

Aggregation, also referred to as flocculation, is a process whereby colloidal particles agglomerate together and come out of suspension to form floc (Wallace & Knight, 2006). This process is dependent on the neutralization of surface charges on particles that repel each other and can easily be disturbed by an increase in the shear forces exerted on particles, e.g. with mixing (Wallace & Knight, 2006). In VSB CWs, these shear forces are reduced by the bed media, and in FWS CWs they are reduced, albeit to a lesser extent, by plant media (Wallace & Knight, 2006).

Interception

Interception and filtration are further processes by which CWs remove suspended solids from water. In FWS CWs the effect is low as the root system of hydrophytes is typically not very dense (Norton, 2007). In VSB CWs, this process has much greater effectiveness

due to the bed media acting as a coarse filter (Norton, 2007). There is, however, a compromise to be reached between maximising the effectiveness of VSB CW media acting as a filter, which is generally greater with smaller size media, and minimising the probability of the media clogging, which is reduced with greater particle size (Manios et al., 2003; Norton, 2007). Interestingly, a study by Manios et al. (2003) showed that, in subsurface horizontal flow CWs, there is no significant difference in suspended solids removal between planted and unplanted wetlands.

2.4 Water Quality Testing Methods

To test the effectiveness of a constructed wetland in removing pollutants from influent water, a water characterisation must be performed on influent and effluent stream of the CW. It is infeasible to test water samples for all quality parameters, and therefore it is necessary to choose the most pertinent parameters to test for any given study. These parameters are generally selected based on several factors including the accessibility of the researcher to the testing equipment, costs associated with conducting the tests and the most pertinent water discharge standards of the receiving water body.

2.4.1 pH

pH is defined as the negative logarithm of the concentration of the hydrogen ion, hydron [H^+], shown in Equation 12.

Equation 12: Formula relating concentration of H+ ions to pH

$$pH = -log_{10}[H^+]$$

This concentration of hydrogen ions in moles per litre determines the concentration of the species of most dissolved chemicals in water and is therefore an important parameter that is commonly used to characterize water, and in particular greywater (Eddy et al., 2013).

pH can be measured with a pH meter, an electric device that can measure hydrogen ion activity, or qualitatively using universal pH paper (Eddy et al., 2013). pH meters generally consist of a voltmeter attached to a hydron-responsive electrode and a reference or control electrode (*PH Meter*, 2021). When immersed in a solution, these electrodes create a battery and the resulting electrical potential that is generated between the two electrodes can be measured and converted to pH (*PH Meter*, 2021). Owing to the accuracy, availability and inexpensiveness of portable pH meters, they are generally used for pH measuring requirements.

It is important to highlight the difference between pH and alkalinity, as these two parameters are commonly conflated. It is often wrongly assumed that a high pH is indicative of an alkaline solution. Although an alkaline solution does have a high pH, it is also possible for a solution with high pH to have low alkalinity. The alkalinity of a solution is a measure of the concentration of hydroxides [OH^-], carbonates [CO_3^{2-}] and bicarbonates [HCO_3^-] and must be measured separately by titration with an indicator such as methyl orange/purple and/or phenolphthalein (Eddy et al., 2013).

In wastewater, the pH of the water supply (i.e. the municipal water supply in the area) is one primary contributing factors that determines the pH of greywater, with typical values reported between 5-9 pH (Oteng-Peprah et al., 2018). Alkaline chemicals, such as sodium-hydroxide based soaps, used in detergents will produce a higher pH in greywater which chiefly arises from laundry effluent, whilst lower pH values can be attributed to the presence of organic acids (Oteng-Peprah et al., 2018).

2.4.2 Electrical Conductivity

The electrical conductivity of water is another standard parameter that is commonly measured in water quality analyses. As the name suggests, is it a measure of the ability of the water to conduct electricity. Electrical conductivity is generally measured with a probe that consists of two electrodes across which a voltage is applied (*5.9 Conductivity | Monitoring & Assessment | US EPA*, 2012). The drop in voltage between the two electrodes caused by the resistance of the water is then measured and use to calculate electrical conductivity per centimetre (*5.9 Conductivity | Monitoring & Assessment | US EPA*, 2012). The SI units for electrical conductivity in water are millisiemens per meter (mS/m) (Eddy et al., 2013). For conductivity measurements to be accurate and representative, the readings must also include the temperature of water at the time of measuring conductivity (*Conductivity, Salinity & Total Dissolved Solids*, n.d.). This is due to the temperature dependence of electrical conductivity in water, with higher temperatures increasing ionic mobility and the solubility of many minerals and salts (*Conductivity, Salinity & Total Dissolved Solids*, n.d.). In general, for every 1ºC increase, the electrical conductivity of the solution will increase 2-4% (*Conductivity, Salinity & Total Dissolved Solids*, n.d.).

Electrical conductivity is an important parameter to measure because it gives an indication of the concentration of ions in the water and can be used as a surrogate measure for total dissolved solids (Eddy et al., 2013). Furthermore, electrical conductivity, and by extension

total dissolved solids, is one of the most common qualitative parameters that municipalities in South Africa measure when taking water samples to ensure conformity with discharge standards. This is often due to the ease and cost-effectiveness of measuring electrical conductivity which makes it a practical parameter to measure on a regular basis.

In general, total dissolved solids (TDS), can be estimated from electrical conductivity (EC) via the relationship shown in Equation 13.

Equation 13:Generalised formula for converting total dissolved solids to electrical conductivity

$$TDS\ (mg/l) \cong EC\ (\mu S/cm)\ \times (0.55 - 0.70)$$

However, this relationship does not hold for high strength wastewater or industrial effluent, where the ratio of electrical conductivity to TDS can often be more than 0.7 (Eddy et al., 2013).

Electrical conductivity of greywater has been reported to fall within the range of 14 and 3000 µS/cm (Ciabatti et al., 2009; Prathapar et al., 2005). Higher electrical conductivity has been observed in areas with water scarcity and groundwater seepage as a result of greywater containing higher concentrations of dissolved solids and ions, especially when these areas are near to the sea and thus have more saline groundwater (Oteng-Peprah et al., 2018). In addition, leaching of old or poorly maintained plumbing materials in greywater can also increase its electrical conductivity (Oteng-Peprah et al., 2018). Oteng-Peprah et al. (2018), reported a statistically significant difference between electrical conductivity of greywater from in-house sources and outside sources, with outside sourced greywater being higher. This is most probably due to water from outside sources being used multiple times in the home before being discarded and therefore accumulating dissolved ions (Oteng-Peprah et al., 2018).

2.4.3 Oxidation-Reduction Potential

Oxidation-reduction potential (ORP) is a measure of the strength of oxidising or reducing conditions in water (Myers, 2019). Based on the chemical compounds that are present in water, the conditions will be either oxidative (ORP > 0), reductive (ORP <0) or, less commonly, neither oxidative nor reductive (ORP = 0) (Myers, 2019). ORP is only a measure of the intensity of the oxidative or reductive conditions and not the concentration of the ions that create those conditions, and for this reason is measured in mV rather than mg/l (Myers, 2019).

ORP probes contain two electrodes: one reference electrode made from silver chloride or saturated calomel which is connected to a salt water solution with a half-cell potential of 0.0 mV, and a sensing or measuring electrode made from gold, graphite or platinum (*Oxidation Reduction Potential (ORP)*, n.d.)

Beyond giving an indication of the intensity of the oxidative or reductive conditions in water, ORP is an important parameter to measure in wastewater as it indicated the type of biochemical reactions which may be taking place, and therefore the chemical transformations that can be expected, as shown in Table 9.

Table 9 Biochemical Reactions and Corresponding ORP values. Adapted from: YSI (2021)

Biochemical Reaction	**ORP range (mV)**
Nitrification	+100 to +350
cBOD degradation with free molecular oxygen	+50 to +250
Biological phosphorus removal	+25 to +250
Denitrification	+50 to -50
Sulfide (H_2S) formation	-50 to -250
Acid formation (fermentation)	-100 to -225
Methane production	-175 to -400

2.4.4 Chemical Oxygen Demand

Chemical Oxygen Demand (COD), is a measure of the organic material in wastewater that can be oxidised chemically (Sawyer et al., 2002). COD measurement most commonly uses an acidic potassium dichromate solution (which is orange in colour) as the strong oxidant that is required in the measurement process, which is then reduced to chromium salts (green in colour) in the oxidation process (Kosseva, 2013). The reagent solution of potassium dichromate also contains silver sulfate as a catalyst and mercury sulfate as a chloride inhibitor/mask. The reacted solution then undergoes absorbance measurement using a colorimeter or spectrophotometer (Kosseva, 2013).

Chemical Oxygen Demand can be fractionated in many sub-categories including total COD, settled COD, filtered COD, soluble COD, particulate COD, readily biodegradable soluble COD, slowly biodegradable colloidal and particulate (enmeshed) COD, nonbiodegradable soluble COD, nonbiodegradable colloidal and particulate COD (Eddy et al., 2013). The most common categories for the measurement COD are total COD, soluble

COD and settled COD. Differentiating between the different types of COD is especially important for wastewaters with high total suspended solids (TSS) as the measurement results can vary greatly depending on whether the COD of the suspended solids is included or excluded in the total reading (Jingsheng et al., 2006).

Chemical oxygen demand is a pertinent parameter to measure in wastewater for several reasons. Knowing the concentration of COD in wastewater can assist in determining the short term impact of the wastewater on receiving waters like rivers and dams (Chemical Oxygen Demand (COD) - Water Quality Parameter | Hach, n.d.). Because downstream water bodies often contain flora and fauna that are dependent on dissolved oxygen in water for survival. Therefore, controlling the maximum COD of wastewater that is discharged into such bodies can ensure that dissolved oxygen concentrations are maintained at levels that are not harmful to the environment (Chemical Oxygen Demand (COD) - Water Quality Parameter | Hach, n.d.). COD can also be used to estimate biological oxygen demand (BOD) (Chemical Oxygen Demand (COD) - Water Quality Parameter | Hach, n.d.). This is useful because the BOD test typically requires 5 days to obtain results whereas the COD test requires approximately only 2.5 hours (Eddy et al., 2013).

2.5 Application of Constructed Wetlands in informal settlements

Although there have been many studies investigating the effectiveness and sustainability of CWs from a technical and scientific perspective, few make mention of the involvement of the community in the maintenance and proper use of the CW. In a review of the sustainability of using CWs for wastewater treatment, Wu et al. (2015) stated that although CWs offer a low-energy, land-intensive and less-operational-requirements alternative to conventional treatment systems, particularly for remote or underserviced communities, the sustainable operation and successful application of these systems remains a challenge (Wu et. al 2015). Their paper highlighted several parameters that must be considered when designing sustainable CWs including plant tolerance to wastewater, substrate medium, hydraulic load and retention time, however no mention was made of the involvement and role of the community in using the wetland correctly and in maintaining it (Wu et. al 2015). A different review by Zhang et al. (2014), who investigated recent developments in the application of CWs for wastewater treatment in developing countries between the years 2000 and 2013, also focussed heavily on the scientific parameters of wetland design and sustainability. Although the study did state that CWs generally have much lower operation and maintenance costs than conventional wastewater treatment plants, details regarding

whether the maintenance was undertaken by the community itself or by an external 3rd party, were not present in the study (Zhang et al. 2014).

A study by Liu et al. (2009) of recent developments and future challenges of CWs in China, indicated that beyond the advancement in scientific understanding of CWs for wastewater treatment, 'land availability, institutional limitations, and public education will be ongoing challenges for the development of CW technology' (Liu et al. 2009). Building on this theme, Massoud et al. (2009) suggest that employing inappropriate technology and failing to consider the local conditions of the target community can result in project failure which is often mistakenly attributed to a lack of scientific understanding or financial resources. The study undertaken by these researchers goes on to state that the 'Most Appropriate Technology' for implementation in a community is not only the one that can meet requirements from a technical perspective but also one that is economically affordable, socially acceptable and environmentally sustainable (Massoud et al. 2009). They further recommend that the 'community should be able to finance the implementation of the system, the operation and maintenance including the capital improvement needed in the future and the necessary long-term repairs and replacements (Massoud et al 2009). Given the socio-economic reality of Alexandra township lacking many basic services, this is even more important.

3 Methods and Materials

3.1 Study Area

The physical area that was chosen in which to undertake this research is the township of Alexandra, in Johannesburg, South Africa. Figure 3 shows the location of Johannesburg within the Gauteng Province in South Africa. Figure 4 shows the location of Alexandra within the Gauteng Province. Figure 5 shows an arial view of Alexandra with pins JR1, JR2, CW1 and CW2 showing the location of the upstream sampling point on the Jukskei river, the downstream sampling point on the Jukskei river, the location of constructed wetland 1 and constructed wetland 2 respectively. To give further context on the area of study, geographical information for Alexandra from the 2011 census in South Africa is provided in Table 10.



Figure 3 Map of South Africa showing Gauteng province outlined

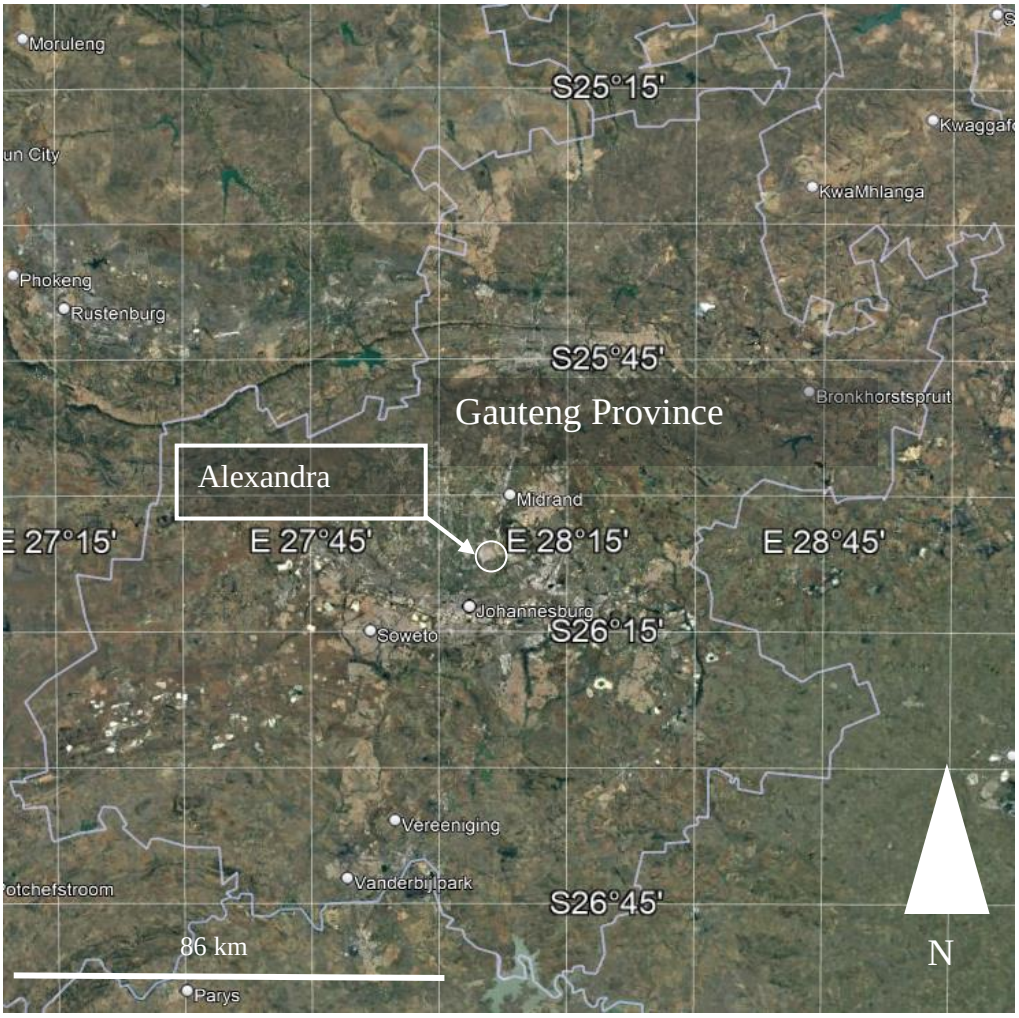

Figure 4 Map of Gauteng province showing Alexandra circled

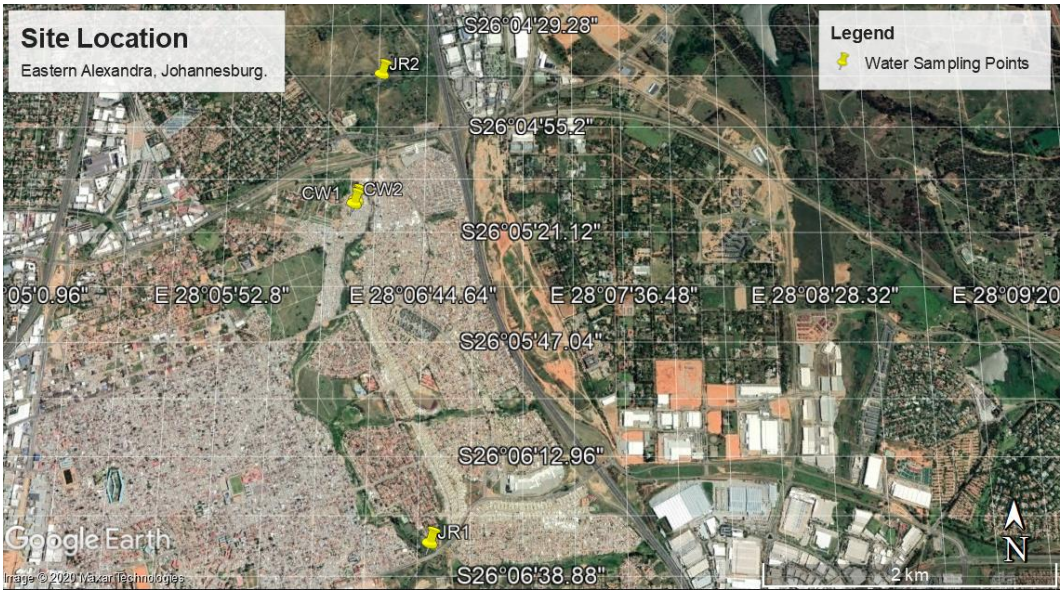

Figure 5 Map showing Alexandra and surrounding areas with pins showing water sampling points and location of constructed wetlands

Area	6.91 km^2
Population	179 624 (25 979/km^2)
Households	63 737 (9218/km^2)
Gender Proportion	52% Male, 48% Female
Racial Composition of Population	99% Black
First Language Composition	26% isiZulu, 23% Sepedi, 11% Xitsonga, 10% isiXhosa, 10% Setswana, 7% Sesotho, 5% Tshivenda

Table 10 Geographical Information for Alexandra (Census 2011: Main Place: Alexandra,)

There are several reasons that Alexandra was chosen as an area of study. Firstly, Alexandra is situated close to the University of Witwatersrand (within 15 minutes' drive, without traffic) and thus makes regular trips between the laboratories at the university and the study site logistically simple. This also increases the accuracy of any laboratory results (which will depend on access to laboratories in the University of Witwatersrand and the South African lockdown regulations) as the time between sample taking and analysis is decreased.

Alexandra also includes areas of informal settlement (consisting of metal shacks and simple brick buildings), the main locus of study in this proposal, which are largely devoid of any formal reticulation and treatment systems for sewage, greywater and rainwater. This makes it a suitable site for the employment of constructed wetlands in greywater treatment.

Another element that contributes to Alexandra's suitability to the proposed research is the existence of a river, the Jukskei, which runs through the area. Water samples collected from this river can provide information regarding the type and concentration of pollutants arising in Alexandra.

Lastly, in their 2009 study of Alexandra, Owusu Asante and Ndiritu find that here interconnected problems such as overpopulation, overcrowding, the area's soil which is largely composed of silt and rock, its vulnerability to stormwater runoff as well as floods, mixing of sewage and greywater, high unemployment and low living standards, and non-payment for state services like water, all come together to create a context in dire need of intervention.

The context of Alexandra thus corresponds directly with the problem identified through the gap in the literature, and the substantive area of enquiry identified in the research question, making it the perfect fit for the study.

3.2 Design and Construction of Constructed Wetlands

With the CWs in this study being designed for and constructed in the field (the informal settlement in Alexandra) rather than in a laboratory or controlled environment, the design process, construction and commissioning of these CWs gains particular importance. The following sub-sections explore this process and highlight the most important steps and elements of each stage.

3.2.1 Design Philosophy & Design Process

Before designing a CW for use in a community setting, it is vital to understand the underlying dynamics of the community and local environment. Silvertown, Alexandra, consists of mostly shack and simple brick homes that are built extremely close to one another (often sharing the same wall between two different homes). Potable water for drinking, washing, cooking and general home-use is available from low-pressure taps that are located every 30-40m outside of homes, and most residents have to resort to using buckets to transport water to their homes. Furthermore, there is no formal drainage reticulation for stormwater or sewerage, and most of this water flows in shallow informal trenches (shown in Figure 6 and Figure 7) in-between homes until it reaches the Jukskei River (shown in Figure 8). These trenches often get congested with plastic and other debris, and this leads to greywater and blackwater flooding into homes, especially when there is a heavy rain. This highlights the challenge of finding a space sufficiently sized to accommodate a CW, and reticulating effluent into the CW and treated effluent out of the CW to the trenches or river.



Figure 6: Residents in Silvertown, Alexandra washing clothes at a typical communal tap with wastewater flowing into a typical informal trench that leads to the Jukskei River



Figure 7: A typical shallow informal trench in Silvertown, Alexandra that links wastewater from communal tap (flowing into the trench via the PVC pipe on the right) to the Jukskei River



Figure 8: Jukskei River downstream of Alexandra

Furthermore, safety and security are of concern in the area, both for residents and material belongings that are not secured. Metal (from fences, taps, valves etc.) is especially difficult to secure as it is often stolen and sold as scrap metal. Although electricity is available, many of the connections are illegal, which can result in fires and electrical shock, and any equipment that requires electricity (e.g. pumps, sensors etc.) that is left unattended is likely to be stolen.

On the other hand, when a community takes ownership of a project or technology, this can lead to the community ensuring the technology is looked-after and is more secured. This requires that the community is involved in the design, planning, construction and commission of the plant and that their input is sufficiently weighted to ensure the final outcome is aligned with their incentives.

Before designing and constructing the two CWs in this study, several workshops were held with community members and leaders to discuss their views on water sanitation in their neighbourhood. These workshops explored the importance that residents give to having a clean environment that is free from foul-smelling greywater and blackwater, previous attempts by the community and government to address this issue, as well as brainstorming sessions that included opportunities for the community to suggest feasible solutions and to take part in the design process of the CWs. Although these workshops did not form a part of the scope of this researcher's study, the workshops were part of the broader project which this researcher was associated with, and for this reason are mentioned here to give a greater context on the study.

3.2.2 Construction Materials Selection and Procurement

One of the main aims of the project of which this research study was a component was to create CWs for water treatment in informal settlements that could easily be replicated in other informal settlements either formally by the public sector, or informally by initiatives from resident communities themselves. Based on this aim, and the design philosophy communicated above, construction materials for the CWs were selected based on several factors including availability in the local vicinity, cost-effectiveness, durability for several years and aesthetics. As outlined above, the materials selected intentionally excluded metals and any equipment that would require electricity.

The following materials were selected for construction:

- Locally-procured 'recycled' bricks obtained from a supplier that recovers bricks from abandoned or demolished houses. These bricks were used for the containing walls of the CW (shown in Figure 10)
- Locally-procured cement and builder's sand
- Locally-procured coarse gravel for the CW substrate material (shown in Figure 9)
- PVC pipes and elbows from a hardware store. These pipes were used for part of the basic internal reticulation of the CW.
- 'Pool-pipe' procured from a hardware store. This flexible pipe was use as a level control mechanism on the outlet of the CW.
- Drain funnel and grate procured from a hardware store. This was used as the feeding point for the CW.
- Bitumen Paint and Geotextile fabric procured from a hardware store. This was used for the internal waterproofing of the CW.
- Plastic tarp procured from a hardware store. This was also used for the internal waterproofing of the CW when the bitumen paint proved to be insufficient.



Figure 9: Gravel for constructed wetland bed media that was procured from a local supplier in Silvertown, Alexandra



Figure 10: Re-used bricks that were procured from a local supplier in Silvertown, Alexandra for building the containing walls of the constructed wetland

3.2.3 Location Selection

The community workshops mentioned above were often followed by site-surveys during which the researchers would walk around the neighbourhood with the residents to better understand the current challenges of water sanitation in the area, and to identify potential sites where the CWs could be installed.

Several factors were taken into consideration when assessing the suitability of potential site locations. Firstly, it was identified that the CWs would be best placed in close proximity to municipal water taps as residents often wash their clothes and cooking utensils at these taps, and then discard the wastewater nearby on the ground or in an informal drainage trench. Locating the CWs next to these taps would make it easier for residents to dispose of their wastewater in the inlet of the CW. Secondly, issues pertaining to land ownership and land use seemed to be common in Silvertown and therefore engaging the local residents and obtaining their confirmation and approval of potential sites was found to be important. Furthermore, because the containing walls of the CW would protrude 30-50cm above ground level, there was also potential to engage with residents regarding the directionality of the CW, to allow for this bund wall to simultaneously act as a stormwater diverter and a walking path above the informal drainage trenches.

With the above points in mind, 4 potential sites, Site 1, Site 2, Site 3 and Site 4, were initially identified for the constructions of the CWs, with the aim of selecting two. These sites are pictured below in Figure 11, Figure 12, Figure 13 and Figure 14 respectively.



Figure 11: Site 1 that was identified for the constructed of a constructed wetland (perimeter outlined in red). Communal tap with resident filling bucket shown on right. (Coordinates: 26°05'17.0"S 28°06'25.3"E)



Figure 12: Site 2 that was identified for the constructed of a constructed wetland. Communal tap shown to left with resident assisting to take measurements with yellow measuring tape in centre-right. (Coordinates: 26°05'16.1"S 28°06'25.9"E)



Figure 13: Site 3 that was identified for the construction of a constructed wetland (perimeter outlined in red). Communal tap shown at left. (Coordinates:26°05'18.0"S 28°06'26.7"E)



Figure 14: Site 4 that was identified for the construction of a constructed wetland. Communal tap shown on left with resident filling bucket. (Coordinates: 26°05'18.0"S 28°06'27.2"E)

Site 3 was not selected, although it did provide for sufficient space, because the municipal tap it contained was found to not be used regularly at the time.

Site 4 was not selected, although it did include a municipal water tap that was regularly used, because there was not sufficient space for an adequately-sized CW based on the flowrate of wastewater from the tap and the adjacent road on which large trucks would frequently pass.

Thus, Site 1 and Site 2 were selected for the construction of CW1 and CW 2 respectively.

3.2.4 Final Design

Through several design iterations, and considering the design philosophy mentioned in this section, the design in Figure 15 was chosen for CW 1 and the design in Figure 16 was chosen for CW 2. The designs for both CWs are essentially the same, with CW 1 being about 1.2m longer than CW 2, owing to the larger space that was available for CW 1.

This design incorporates an inlet feeding drain with a grate screen which separates larger food particles and large solids from entering the wetland. From the inlet drain, effluent is directed to a settling chamber, to allow for further separation of solids, especially from food particles that are very common in wastewater discharged from buckets in Silvertown, Alexandra. Effluent then overflows out of the settling chamber and through the gravel reed bed where the main biological and treatment mechanisms take place. At the end of the

gravel reed bed is a drain connected to a flexible hose or ‘pool pipe’. This flexible hose allows for the water level within the CW to be regulated and controlled, without the need for valves or electronic equipment. By simply raising or lowering the flexible hose, the water level in the CW can be raised or lowered respectively. Once treated effluent exits from the flexible hose, it then exits the CW via the exit drain located at the end of the CW.

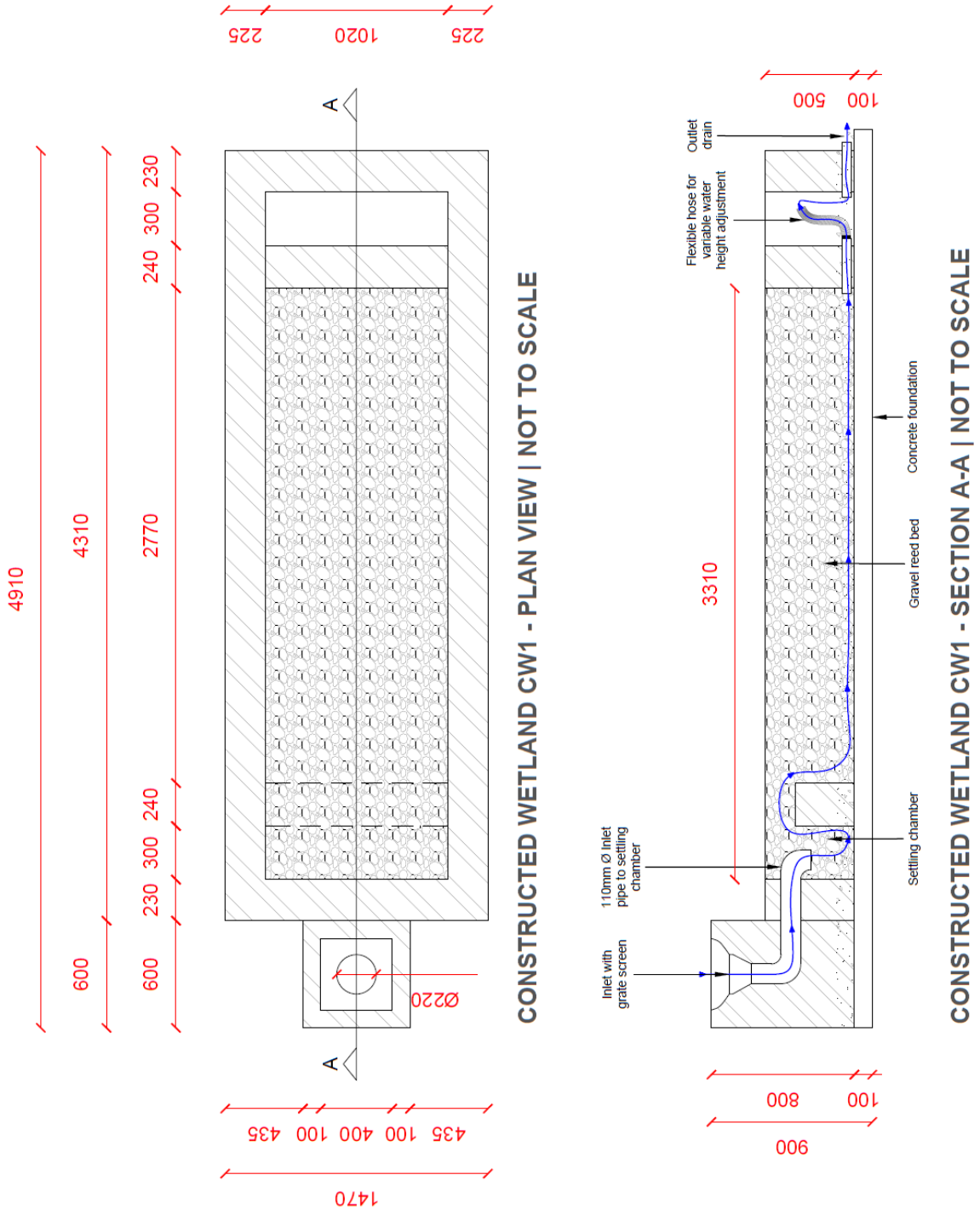

Figure 15: Technical Drawing for Constructed Wetland 1

The initial design for the CWs, consisted of single brick walls for external and internal walls. However, this was later changed to double brick walls, as shown in Figure 15 and Figure 16, based on the builder's level of experience and recommendations. The technical drawings in Figure 15 and Figure 16 were drawn by this researcher based on the final dimensions of the completed CWs that were built in Alexandra.

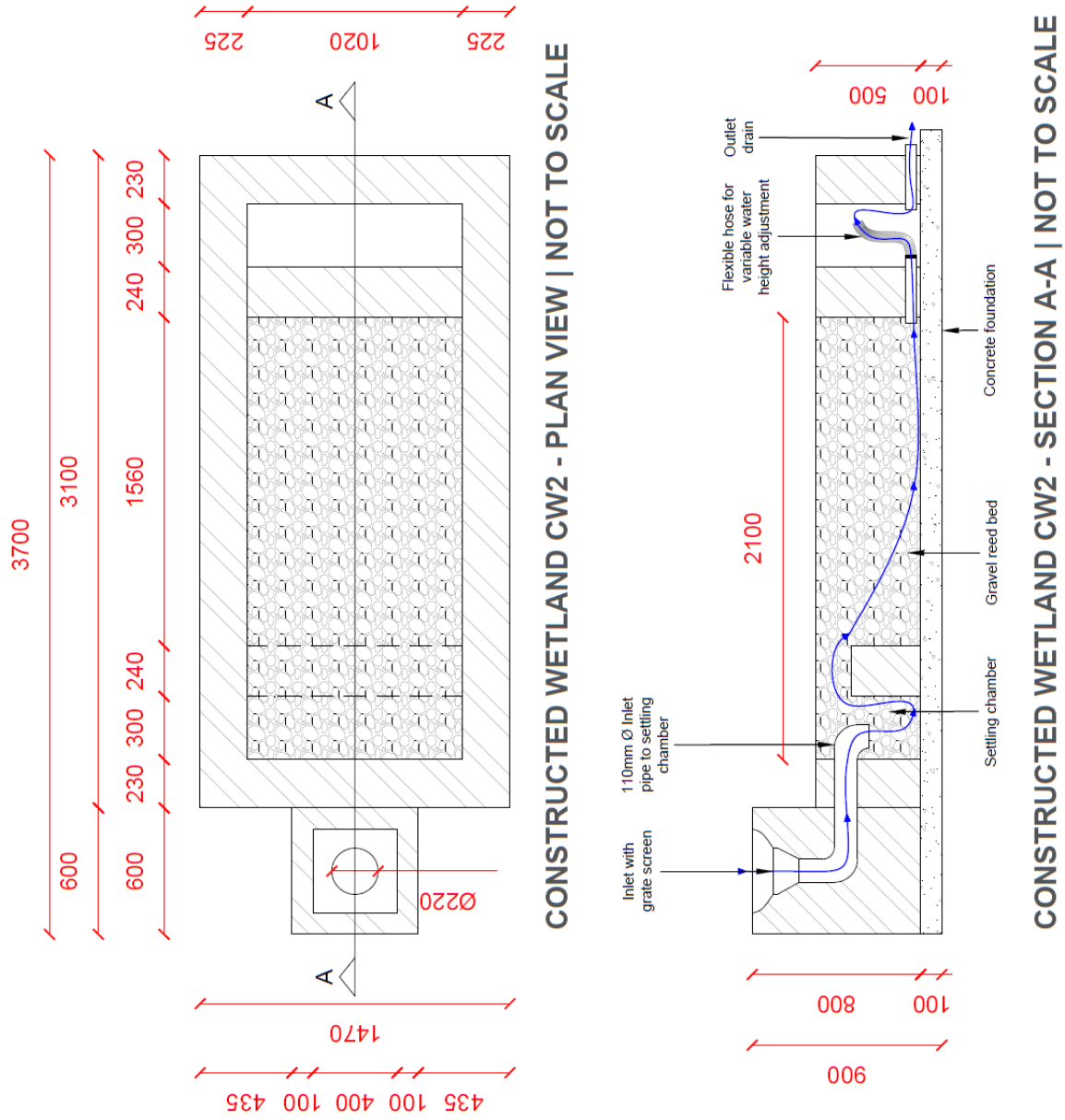

Figure 16: Technical Drawing for Constructed Wetland 2

To share a sense of the building process in Silvertown, Alexandra, some photos are shared in this section. Figure 17 shows the initial part of the construction process of CW 1 which involved laying the concrete foundation and retaining walls. Figure 18 shows an almost complete CW 2 before filling with gravel and the construction of the inlet feeding drain.

Figure 19 shows the flexible hose outlet drain design for the CWs that allows the inside water level of the gravel reed bed to be adjusted without the need for any valves or electrical equipment. Figure 20 shows workers and community members planting *Juncus* reeds in a completed CW 1.



Figure 17: Builders laying foundation and retaining walls for Constructed Wetland 1



Figure 18: Constructed wetland 2 showing main retaining walls completed and waterproofed



Figure 19: Exiting drain system of constructed wetland showing water flowing through flexible drainage hose



Figure 20: Constructed wetland 1 completed with builders and residents planting reeds

3.3 Methods and Materials for Water Quality Monitoring

To assess the effectiveness of the CWs in removing and/or transforming pollutants in the wastewater, water characterisations were conducted over the course of three months. These characterisation tests were conducted on both influent and effluent (with respect to the CW) stream, so that pollutant concentrations could be compared and analysed. In addition, water samples were also collected and analysed from the Jukskei River at a point that is upstream and downstream from Alexandra to better understand the effect of anthropogenic pollution on the river.

3.3.1 Sampling Locations

Water samples were collected from the inlet and outlet of CW 1 and CW 1 as well as from the Jukskei River at a point upstream and a point downstream of Alexandra. The exact locations of the upstream and downstream Jukskei river sampling points were determined based on accessibility to the river at those locations. Figure 21 shows the inlet region of CW 1 from which samples were taken using a syringe connected to a thin pipe to extract water from below the gravel level. Figure 22 shows the outlet of CW 1 from which samples were collected by lowering the flexible ‘pool pipe’ to allow water to flow out of the pipe and into the sampling jar. Figure 23 shows the inlet region of CW 2 from which samples were collected by using a syringe and thin pipe extract water from the ‘cylindrical trellis’ access point. Figure 24 shows the outlet region of CW 2 from which samples were collected by lowering the flexible ‘pool pipe’ to allow water to flow from the pipe into the sampling jar. Figure 25 shows the Jukskei River sampling point that is upstream of Alexandra. At this sampling point, samples were collected directly from the river with the sampling jar. Figure 26 shows the Jukskei River sampling point that is downstream from Alexandra. At this sampling point samples were collected directly from the river with the sampling jar.



Figure 21: Inlet region of constructed wetland 1 showing large PVC elbow from inlet drain on the right. Photo taken on 23.09.2020 which was the second week of sampling after the national COVID-19 lockdown in South Africa, and therefore the vegetation in the constructed wetland had not yet grown back. (Coordinates: 26°05'17.0"S 28°06'25.3"E)



Figure 22: Outlet drain of constructed wetland 1 showing the blue flexible 'pool pipe' that connects the internal drain in the constructed wetland to the external drain. (Coordinates: 26°05'16.8"S 28°06'25.3"E)



Figure 23: Inlet region of constructed wetland 2. To the left is the large white PVC elbow that connects the incoming drain to the wetland, and in the centre is the black 'cylindrical trellis' used to access inlet water in the wetland. (Coordinates: 26°05'16.1"S 28°06'25.9"E)



Figure 24: Outlet drain of constructed wetland 2. The blue flexible 'pool pipe' connects the internal drain in the wetland to the external drain which drain into an informal trench that leads to the Jukskei River. (Coordinates: 26°05'16.1"S 28°06'26.1"E)



Figure 25: Jukskei River sampling points upstream of Alexandra located under the bridge on London Road. (Coordinates: 26°06'35.1"S 28°06'46.8"E)



Figure 26: Jukskei River sampling points downstream of Alexandra located in a site that is property of the University of Witwatersrand (Coordinates: 26°04'43.4"S 28°06'39.7"E)

Table 11: Summary of sampling points and coordinates for each.

Sampling Point (sample point code)	Coordinates
Constructed Wetland 1 Inlet (CW 1 In)	26°05'17.0"S 28°06'25.3"E
Constructed Wetland 1 Outlet (CW 1 Out)	26°05'16.8"S 28°06'25.3"E
Constructed Wetland 2 Inlet (CW 2 In)	26°05'16.1"S 28°06'25.9"E
Constructed Wetland 2 Outlet (CW 2 Out)	26°05'16.1"S 28°06'26.1"E
Jukskei River Upstream of Alexandra (Jukskei Upstream)	26°06'35.1"S 28°06'46.8"E
Jukskei River Downstream of Alexandra (Jukskei Downstream)	26°04'43.4"S 28°06'39.7"E

3.3.2 Sampling Frequency

Sampling was done on a weekly basis for a rolling 12-week period. Table 12 provides details of the various samples that were collected and tested on each sampling date.

Table 12: Table showing samples that were collected on each sampling date from the various sampling points. Samples marked with '*' were not collected due to security concerns of collecting samples from the Jukskei in Alexandra.

	Sample Collected (Y/N)					
Date	Jukskei Upstream of Alexandra	Jukskei Downstream of Alexandra	Inlet of Constructed Wetland 1	Outlet of Constructed Wetland 1	Inlet of Constructed Wetland 2	Outlet of Constructed Wetland 2
16.09.2020	Y	Y	Y	Y	Y	Y
23.09.2020	N*	Y	Y	Y	Y	Y
30.09.2020	N*	Y	Y	Y	Y	Y
08.10.2020	N*	Y	Y	Y	Y	Y
14.10.2020	Y	Y	Y	Y	Y	Y
22.10.2020	Y	Y	Y	Y	Y	Y
29.10.2020	N*	Y	Y	Y	Y	Y
05.11.2020	N*	Y	Y	Y	Y	Y
12.11.2020	Y	Y	Y	Y	Y	Y
19.11.2020	Y	Y	Y	Y	Y	Y
26.11.2020	N*	Y	Y	Y	Y	Y
03.12.2020	N*	Y	Y	Y	Y	Y

3.3.3 Sampling Challenges

Several challenges were faced in the collection of water quality measurements which must be taken into account when analysing the data available. Firstly, the national COVID-19 lockdowns legislated by the South African government resulted in the research not being able to take water samples from the CWs until approximately 6 months after the wetlands were constructed. This meant that it was not possible to collect water quality data from the start up water treatment phase of the CWs. Secondly, with respect to the laboratories at the University of Witwatersrand, although lab personnel made great effort to assist as much as they could, there were changes in staff and changes in computer systems which resulted in some of the results from ion chromatograph not being recorded for certain samples.

3.3.4 Sampling Techniques

Sampling technique is an important process to follow, and results can vary widely from measurement to measurement if a standard method is not followed. The sampling procedure used in this study was as follows:

- Sterilised glass vessels with securable lids that had been washed and rinsed thoroughly with deionised water were used as the sample vessels.
- Each sample vessel was rinsed three times with the water that was to be sampled before finally filling the sample vessel and securing the lid tightly.
- The sample vessels were stored in a cooler box to maintain a consistent and low temperature of the vessels between sampling in the field and analysing in the laboratory.
- The design of the CW was such that, for some sampling points, the sampling vessel could be filled directly from the sampling points, but for others, a syringe with an elongated pipe was required to siphon the sample water out from beneath the gravel to then be filled into the sampling vessel (pictured in Figure 27).
- Nitrile gloves were used in the sampling procedure to maintain hygienic standards and reduce any possible contamination from the sampler's hands (especially for microbiological studies).
- Results for measurements taken in the field and laboratory were recorded in the researcher's field diary and later transcribed into Microsoft Excel.

3.3.5 Measured Parameters

In consultation with the research team and the project technical team, it was agreed that the following parameters would be monitored:

1. pH
2. Electrical Conductivity
3. Oxidation-Redox Potential
4. COD
5. Sulfate
6. Nitrite
7. Nitrate
8. Phosphate
9. Fluoride
10. Chloride
11. Total Coliforms
12. *E. Coli*

The selection of these parameters for monitoring and analyses was based on the consideration of several key factors including feasibility, cost, ease of measuring, access to equipment and apparatus and restraints imposed by the COVID-19 'lockdowns' in South Africa. Further considerations were of physical security during the sampling process in Alexandra, and in certain situations samples were not collected as it was deemed unsafe to do so.

The following section will outline the key parameters that were measured, the basic definition of each of these parameters, the importance in measuring of the parameters, and general findings from literature regarding these parameters in greywater.

The treated water parameters measured (effluent of the CW) would generally be compared to the untreated water parameters measured (influent to the CW) to assess the effectiveness of the CW in treating effluent. The treated water parameters could also be compared to the legislated water discharge standards to a receiving water body, as defined by the Department of Water and Environmental Affairs (currently named the Department of Water and Sanitation at the time of writing this dissertation). Table 13 shows the wastewater limit values, for the same or similar parameters measured in this study, applicable to discharge of wastewater into a water resource (Department of Water and Environmental Affairs of South Africa, 2013).

Table 13 Wastewater limit values applicable to discharge of wastewater into a water resource

Substance/Parameter	**General Limit**
Faecal Coliforms (CFU per 100ml)	1000
Chemical Oxygen Demand (mg/l)	75
pH	5.5 – 9.5
Electrical Conductivity (mS/m)	70 mS/m above intake to a maximum of 150mS/m
Ortho-Phosphate as Phosphorus (mg/l)	10
Fluoride (mg/l)	1
Nitrate/Nitrite as Nitrogen (mg/l)	15

pH

In this study, the Hanna multiparameter HI98195 portable waterproof meter was used to measure the pH of the water samples. This meter has a range of 0.00 to 14.00 pH, a resolution of 0.01 pH and an accuracy of ± 0.02 pH (*HI98195 Multiparameter PH/ORP/EC/Pressure/Temperature Waterproof Meter*, 2013). In conducting tests on the water samples, the probe of the meter was placed in the water samples to sufficient depth and the reading was allowed to stabilise before recording the measurement.

The calibration procedure followed for this multiparameter meter as per the instruction manual consisted of the following steps:

1. Small quantities of the buffer solution were poured into two small beakers. One beaker was used for rinsing the probes and sensor and the other was used for calibration.
2. The pH probe and temperature sensor were immersed in the first buffer rinse solution beaker and stirred gently.
3. The pH probe and temperature sensor were then immersed in the second calibration buffer solution.
4. Once the reading had become stabilized, the multiparameter meter was calibrated for the value of the buffer solution in use.

This calibration procedure was also used for calibrating electrical conductivity on the probe.

Electrical Conductivity

In this study, the Hanna multiparameter HI98195 portable waterproof meter was used to measure the electrical conductivity of the water samples. This meter has a range of 0 to 200 mS/cm (absolute EC up to 400 mS/cm), a resolution of up to 0.001 mS/cm and an accuracy of ± 1% of the reading (*HI98195 Multiparameter PH/ORP/EC/Pressure/Temperature Waterproof Meter*, 2013). Furthermore, this meter includes automatic temperature compensation for electrical conductivity readings so that a compensation formula does not need to be applied to the final values recorded (*HI98195 Multiparameter PH/ORP/EC/Pressure/Temperature Waterproof Meter*, 2013, p. 191). In conducting tests on the water samples, the probe of the meter was placed in the water samples to sufficient depth and the reading was allowed to stabilise before recording the measurement.

Oxidation-Reduction Potential

In this study, the Hanna multiparameter HI98195 portable waterproof meter was used to measure the oxidation-reduction potential of the water samples. This meter has a range of -2000.0 mV to +2000.0 m V, a resolution of 0.1 mV and an accuracy of ± 1.0 mV (*HI98195 Multiparameter PH/ORP/EC/Pressure/Temperature Waterproof Meter*, 2013). In conducting tests on the water samples, the probe of the meter was placed in the water samples to sufficient depth and the reading was allowed to stabilise before recording the measurement.

Chemical Oxygen Demand

In this study, Merck Supelco® COD solutions A+B were used as the potassium dichromate solution and the Merck Spectroquant® was used as a photometer. COD tests on the water samples gathered from the CWs were carried out according to the following procedure:

1. The collected water sample was homogenised by vigorously shaking the sample jar for several seconds.
2. The required volumes of COD Solution A and COD Solution B (2.20ml and 1.80ml respectively for the 500-10 000 mg/l COD measuring range) were pipetted into a clean cell using an autopipette and swirled in a test tube oscillator for ~10 seconds to suspend any sediment in the bottom of the cell.
3. An autopipette was then used to add 1.0 ml (in the case of measuring in the 500 – 10 000 mg/l COD range) of the water sample to the cell, being careful to allow the sample to run down the side of the tilted cell onto the reagent.
4. The screwcap of the cell was then tightly fitted, and the cell was vigorously mixed on a test tube oscillator for a further ~20 seconds.
5. The cell was then heated at 148ºC for 120 min in a thermo-reactor.
6. Following heating, the cell was then allowed to rest for 10min to cool before it was swirled again and then allowed to cool to room temperature.
7. Once at room temperature, the cell was then placed in the photometer and 3 separate readings were taken.

Ions

Analyses of wastewater should also include the measurement of the concentration of various ions in water. The difference in concentrations of these ions can then be compared between the inlet and outlet of the wastewater treatment system (in this case CWs) to assess the effectiveness of the system in pollutant removal. Ion concentration analyses also assist in better understanding the various chemical and biochemical reactions that are taking place within the system.

In this study, the concentrations of the following ions were measured:

1. Sulfate (SO_4^{2-})
2. Nitrite (NO_2^-)
3. Nitrate (NO_3^-)

4. Phosphate (PO_4^{3-})
5. Fluoride (F^-)
6. Chloride (Cl^-)

An ion chromatography analyser was used to measure the concentration of these ions. The analyser was first calibrated with standard solutions of 5ppm, 10ppm, 15ppm, 20ppm and 50ppm before measuring the ion concentrations of the samples. These standard solutions were prepared using an autopipette, a one-molar standard solution for each of the ions, and deionised water as the dilution medium.

Coliform Bacteria

Coliform bacteria are often monitored in wastewater streams because their presence is an indicator of faecal contamination (World Health Organization, 2006). The most reliable group of coliforms that is a faecal contamination indicator is *Escherichia coli* (*E. coli*) as it originates almost exclusively from faeces (World Health Organization, 2006).

In this study, the IDEXX Quanti-Tray system was used to measure bacterial counts of both total coliforms and *E. coli*. The test procedure began by diluting samples in 100 ml water that was deionised and sterilised in an autoclave. Because concentrations of coliforms in this wastewater were very high, the dilution ratios used were also high and ranged from 0.5 µl in 100 ml to 5 µl in 100 ml which equates to 0.0005% and 0.005% dilution rations respectively. The Colilert® reagent powder was then added to the diluted sample in a sterilised sample vessel, the lid secured, and the vessel shaken rigorously until the reagent was completely dissolved. The diluted sample-reagent mixture was then poured into a Quanti-Tray/2000 tray, the tray was tapped to release any air bubbles and the tray was inserted upside down into the Quanti-Tray/2000 rubber insert. The rubber insert was then passed through the Quanti-Tray Sealer PLUS (which had been pre-heated) and sealed. The sealed tray was then placed into a preheated incubator at 35ºC ± 0.5ºC for 24 hours. After the incubation period was complete, the tray was removed, and the number of yellow large and small wells was recorded. The tray was then placed under an ultraviolet light in a dark chamber and the numbers of both large and small yellow wells that fluoresced were recorded. These number were then compared to the Quanti-Tray/2000 Most Probable Number (MPN) table to determine the MPN of colony forming units (CFUs) in the diluted sample. This MPN number was then scaled up to account for the dilution factor of the sample. Triplicate samples were run for each sampling point to give a more accurate and

representative value, and therefore the final MPN number reported for each sample is the average of the three tests that were run.



Figure 27: Syringe and glass jars used for sampling. The access point to the water beneath the gravel level is in the black mesh cylinder in the centre of the photograph.

3.3.6 Analytical Techniques

Raw Data Handling

All results from analytical tests were initially inputted into Microsoft Office Excel®. This allowed for general data sorting and manual checking against the initial values that were recorded in the researcher's notebook. Results in Excel were then arranged into a format that is suitable for importing into Tableau® and were imported into the program. Tableau was used primarily as graphing tool to compare various parameters to each other and to produce the graphs presented in the results section of this dissertation.

The results from these tests were plotted in bar graphs in Tableau showing the concentration or value of each parameter for each test during the 12-week sampling period. Due to the inherent challenges experienced in measuring these parameters in the field rather in a controlled laboratory setting, it was not possible to measure results for some parameters for

every location throughout the 12-week sampling period. Where gaps occur in the data, due to samples not being taken because of security concerns or measurements not being available due to the challenges experienced with the COVID-19 pandemic, the sampling date in the bar graph has been left as a blank period rather than a zero value, to differentiate this from actual readings of zero concentration.

Determination of Statistical Significance

There are various statistical analyses that can be conducted to determine the statistical significance of results that are obtained. One of the most common statistical tests for significance is the t-test. There are three main categories of the t-test:

1. The one-sample t-test (sometimes referred to as the student's t-test) is used to decide whether the mean of the results for a given entity is equal to a specific value or not.
2. The two-sample t-test is used to decide whether the means of the results for two different groups are equal or not.
3. The paired t-test is used to decide if the difference between paired measurements of results for a given entity is equal to zero or not.

As the CWs are each a separate entity with paired results for inlet and outlet stream, the paired t-test was used to assess the statistical significance of results from the CWs. The paired t-test was also used to assess the statistical significance of results from the Jukskei River upstream and downstream of Alexandra, as the river is one continuous hydrological system with the upstream and downstream water qualities representing paired results.

The general procedure for calculating the paired t-test from a set of results involves several steps and equations. To begin, the difference between inlet and outlet results is calculated for each sampling date, as shown in Equation 14.

Equation 14: Formula used to calculate difference between inlet and outlet results for the paired t-test

$$Difference\ (d) = Inlet\ Result - Outlet\ Result$$

The differences calculated for the entire sampling period are then used to calculate the mean difference, $\bar{d}$. The standard deviation of differences, S_d, is then calculated, and this is used to calculate the standard error of the mean difference, SE($\bar{d}$), as shown in Equation 15.

Equation 15: Formula used to calculate standard error of the mean difference for the paired t-test

$$SE(\bar{d}) = \frac{S_d}{\sqrt{n}}$$

Where:

SE($\bar{d}$) is the standard error of the mean difference

S_d is the standard deviation of the differences

n is the number of samples

The standard error of the mean difference and the mean difference are then used to calculate the t-statistic, as shown in Equation 16.

Equation 16: Formula used to calculate t-statistic in paired t-test

$$T = \frac{\bar{d}}{SE(\bar{d})}$$

Where:

T is the t-statistic

$\bar{d}$ is the mean difference

SE($\bar{d}$) is the standard error of the mean difference

The t-statistic can then be used to find the p-value on a t-distribution table. The p-value is the final value derived from the paired t-test that determines whether the results are statistically significant. In general, a p-value of less than 0.05 is sought to determine statistical significance, as this indicates a confidence of more 95% that the results are statistically significant. For this reason, the standard p-value of 0.05 was used in this study to determine statistical significance. Furthermore, to reduce the risk of introducing human error and systematize the statistical analysis process, the researcher made use of the paired t-test function in Microsoft Excel to determine the p-values for the results obtained.

Calculation of Average Pollutant Removal Efficiency

When calculating the average pollutant removal efficiency, there are different approaches that can be taken. One approach is to calculate the average influent pollutant concentration and the average effluent pollutant concentration over the period of study, and to then calculate the pollutant removal efficiency based on those two averages. Another approach is to determine the pollutant removal efficiency of each set of samples (for influent and effluent streams), and to calculate the average of values over the period of study. Due to hydraulic retention time of the CWs under study being relatively long (at least several hours

based on observations in the field) and also highly variable depending on the day of the week and time of the day, it was decided to use the former calculation approach as outlined in Equation 17. This equation was only applied to parameters that passed the t-test

Equation 17: Formula used to calculate average pollutant removal efficiency

$$Average\ Pollutant\ Removal\ Efficiency\ (\%) = \frac{\left(\begin{matrix}Average\ Concentration\ of \\ Pollutant\ in\ Influent\end{matrix}\right) - (\ Average\ Concentration\ of\ Pollutant\ in\ Effluent)}{(Concentration\ of\ Pollutant\ in\ Influent)}\ x\ 100$$

To analyse the variability and dispersion of the data, the sample ranges and coefficients of variation were calculated. The coefficient of variation is a useful measure as it is dimensionless, in contrast to standard deviation and variance, and can therefore be used to directly compare the dispersion of different sets of data, even when those sets represent different parameters. The range of the sample was calculated with the standard formula shown in Equation 18.

Equation 18: Formula used to calculate range of data

$$Range\ (X) = Maximum(X) - Minimum(X)$$

The coefficient of variation (CV) of the sample set was calculated in Excel using the formula shown in Equation 19.

Equation 19: Formula used to calculate the coefficient of variation

$$CV = \frac{\sigma}{\bar{x}}$$

Where:

σ is the standard deviation of the samples

$\bar{x}$ is the arithmetic mean of the values

4 Results

This section explores the experimental results obtained from the water characterization tests that were conducted in this study. Results have been grouped by the three systems that were studied, namely CW 1, CW 2 and the Jukskei River.

Details on the analytical approach and statistical analyses conducted are included in Section 3.3.6.

4.1 Constructed Wetland 1

This section explores the weekly results obtained from tests of chemical oxygen demand, *E. coli*, electrical conductivity, oxidation-reduction potential, pH and nutrient ions on influent and effluent water for CW 1. Further details regarding these results can be found in Appendix 1.

4.1.1 Chemical Oxygen Demand

Chemical oxygen demand of the influent and effluent water of CW 1 was measured weekly over the full 12-week period of water sampling and analysis. Results from these tests are shown in Figure 28.

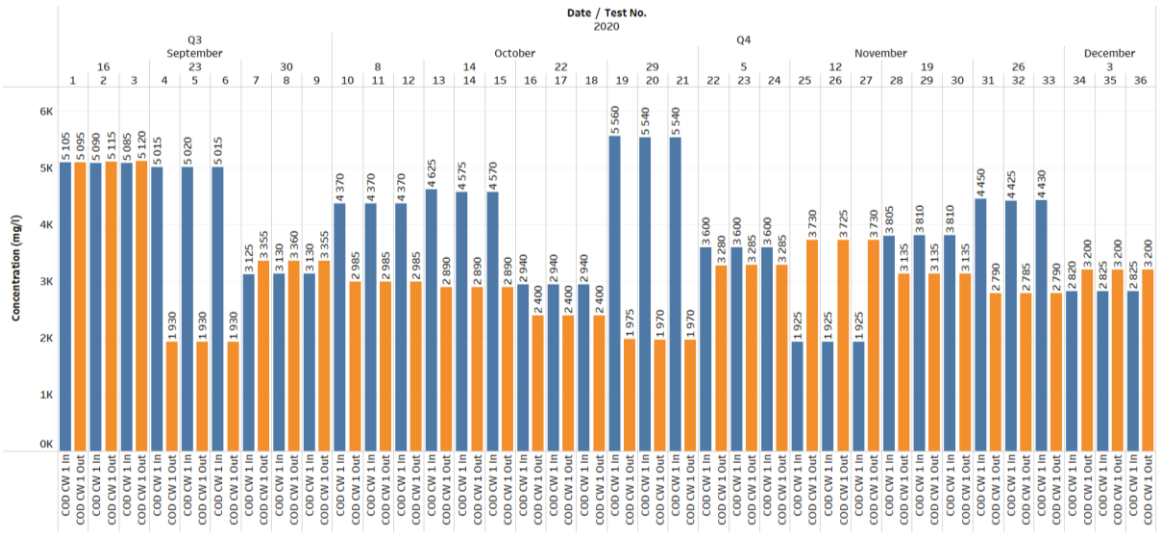

Figure 28: Results from chemical oxygen demand (COD) concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 1

Figure 28 shows results from each of the three spectrophotometer readings that were taken for each sample of water. Each reading number was given a unique test number ranging from 1 to 36 for both inlet and outlet sample readings.

Table 14 shows the statistical summary of the results that are graphed in Figure 28.

Table 14: Statistical summary of results obtained from chemical oxygen demand (COD) concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 1

	COD CW 1 In	COD CW 1 Out
Mean (mg/l)	3937	3063
Min (mg/l)	1925	1930
Max (mg/l)	5540	5090
Range (mg/l)	3615	3160
Coefficient of Variation	27.6	27.4
p-value from paired t-test	0.069	

The difference in the means has a p-value of 0.069 (which is greater than 0.05) which means there is no statistically significant difference between inlet and outlet values. Therefore, these are results are not analysed any further.

4.1.2 Total Coliforms

Total coliforms in the influent and effluent water of CW 1 were measured weekly over the full 12-week period of water sampling and analysis. Results from these tests are shown in Figure 29.

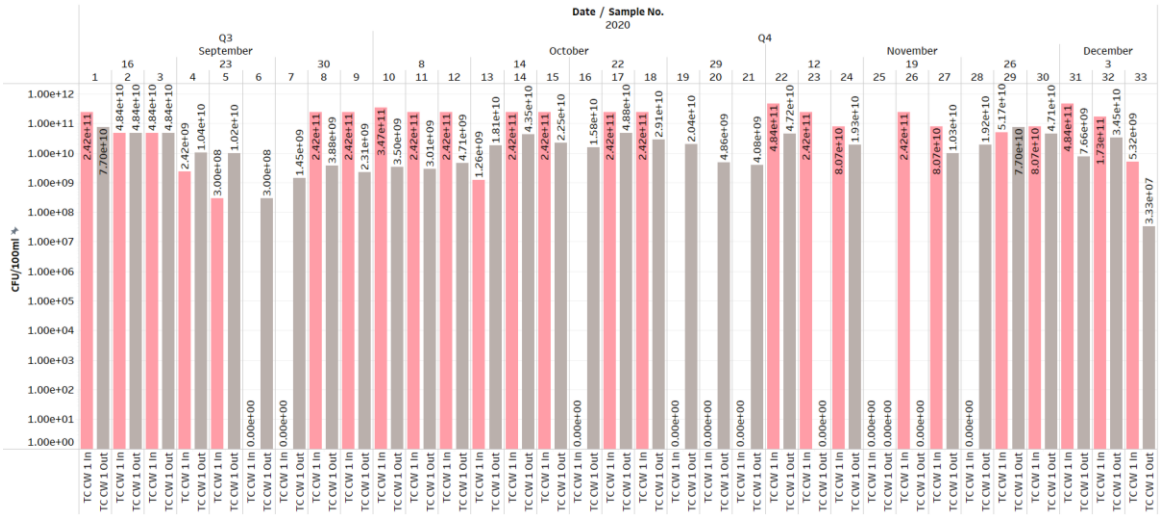

Figure 29: Results from total coliform (CFU/100ml) tests conducted on influent and effluent streams of Constructed Wetland 1

Figure 29 shows results from each of the triplicate samples that were tested for each week. Each of the triplicate samples for inlet and outlet streams was diluted to different concentrations to reduce the likelihood of samples being beyond detection limit after the incubation period, as discussed in Section 3.3.5. Each sample number was given a unique test number ranging from 1 to 33 for both inlet and outlet sample readings. Values that are shown as 0 on the graph were read as <1 CFU/100ml but are shown as 0 to differentiate

between readings of <1 CFU/100ml and samples that were not taken. Readings from sampling week 8 (05 November 2020) were omitted as they were all unreadable due the colour change of the IDEXX kit cells not changing properly from clear to yellow. This could be due to temperature change during the incubation period if someone tampered with the incubator during the 18–24-hour incubation period.

Table 15 shows the statistical summary of the results that are graphed in Figure 29.

Table 15: Statistical summary of results obtained from total coliform (CFU/100ml) tests conducted on influent and effluent streams of Constructed Wetland 1

	Total Coliforms CW 1 In	Total Coliforms CW 1 Out
Mean (CFU/100ml)	1.38E+11	2.07E+10
Min (CFU/100ml)	0	0
Max (CFU/100ml)	4.8392E+11	7.70E+10
Range (CFU/100ml)	4.8392E+11	7.70E+10
Coefficient of Variation	104.4	107.7
p-value from paired t-test	0.0000472	

As can been seen in Table 15, the p-value for this data is far lesser than 0.05 and there is therefore a statistically significant difference between inlet and outlet concentrations of total coliforms in CW 1. Readings of <1 CFU/100ml (shown as 0 in Figure 29) were omitted from the p-value calculation. These results are discussed further in Section 5.

4.1.3 Escherichia Coli

Escherichia coli in the influent and effluent water of CW 1 was measured weekly over the full 12-week period of water sampling and analysis. Results from these tests are shown in Figure 30.

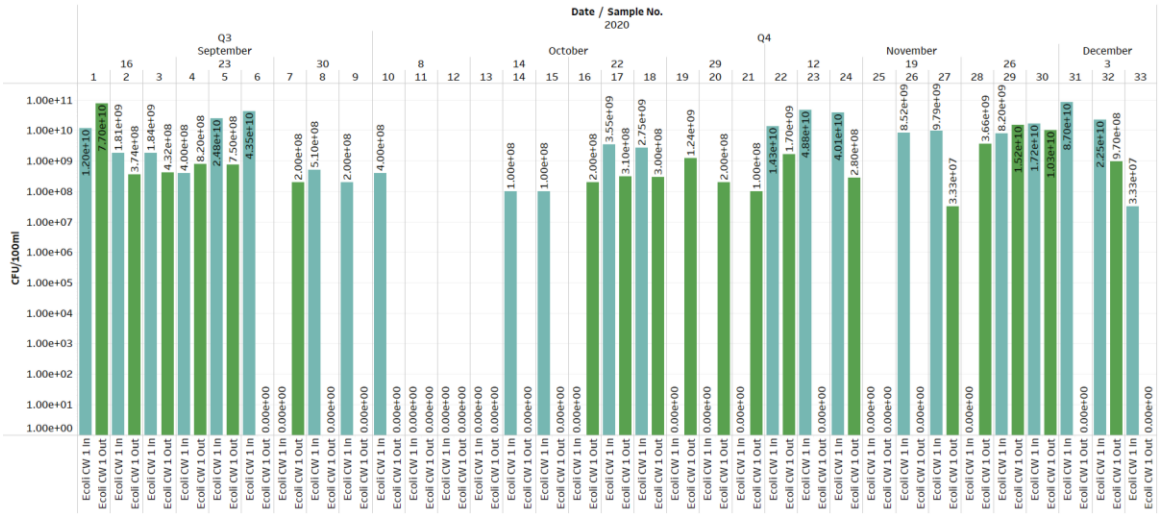

Figure 30: Results from *Escherichia coli* (CFU/100ml) tests conducted on influent and effluent streams of Constructed Wetland 1

Figure 30 shows results from each of the triplicate samples that were tested for each week. Each of the triplicate samples for inlet and outlet streams was diluted to different concentrations to reduce the likelihood of samples being beyond detection limit after the incubation period, as discussed in Section 3.3.5. Each sample number was given a unique test number ranging from 1 to 33 for both inlet and outlet sample readings. Readings from sampling week 8 (05 November 2020) were omitted as they were all unreadable due the colour change of the IDEXX kit cells not changing properly from clear to yellow. This could be due to temperature change during the incubation period if someone tampered with the incubator during the 18–24-hour incubation period.

Table 16 shows the statistical summary of the results that are graphed in Figure 30.

Table 16: Statistical summary of results obtained from *Escherichia coli* (CFU/100ml) tests conducted on influent and effluent streams of Constructed Wetland 1

	***E. coli* CW 1 In**	***E. coli* CW 1 Out**
Mean (CFU/100ml)	1.06E+10	3.45E+09
Min (CFU/100ml)	0	0
Max (CFU/100ml)	8.70E+10	7.70E+10
Range (CFU/100ml)	8.70E+10	7.70E+10
Coefficient of Variation	181.9	392.9
p-value from paired t-test	0.090	

As can been seen from the p-value of 0.090 (which is greater than 0.05) for the data, there was not a statically significant difference between inlet and outlet values, and therefore these results cannot be analysed further to determine pollutant removal efficiency.

4.1.4 Electrical Conductivity

Electrical Conductivity (EC) for influent and effluent water of CW 1 was measured weekly over the 12-week period of water sampling and analysis, as shown in Figure 31.

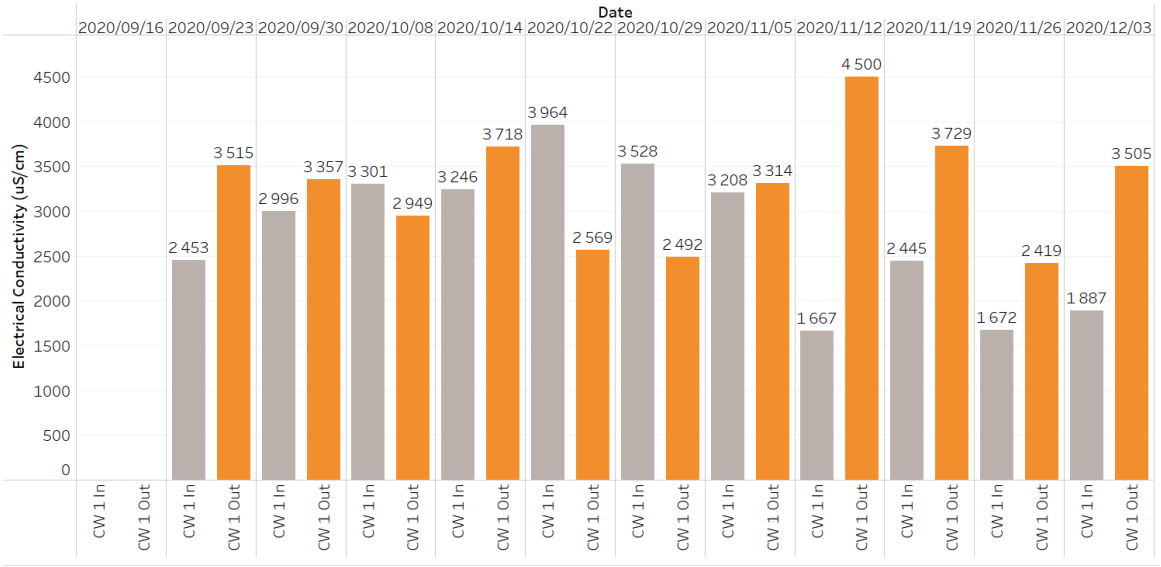

Figure 31: Results from electrical conductivity (µS/cm) tests conducted on influent and effluent streams of Constructed Wetland 1

Results from week 1 (16/09/2020) were not obtained as this was the first week visiting the CWs after the national COVID-19 lockdown in South Africa, and the entry point of CW 1 was not accessible with the EC probe.

Table 17 shows the statistical summary of the results that are graphed in Figure 31.

Table 17: Statistical summary of results obtained from electrical conductivity (µS/cm) tests conducted on influent and effluent streams of Constructed Wetland 1

	EC CW 1 In	EC CW 1 Out
Mean (µS/cm)	2761	3279
Min (µS/cm)	1667	2419
Max (µS/cm)	3964	4500
Range (µS/cm)	2297	2081
Coefficient of Variation	28.4	19.2
p-value from paired t-test	0.18	

The difference in the means has a p-value of 0.18 (which is greater than 0.05) which means there is no statistically significant difference between inlet and outlet values. Therefore, these are results are not analysed any further.

4.1.5 Oxidation-Redox Potential

Oxidation-reduction potential (ORP) was measured in influent and effluent water of CW 1 across the 12-week period of water sampling and analysis. Results from these tests are shown in Figure 32.

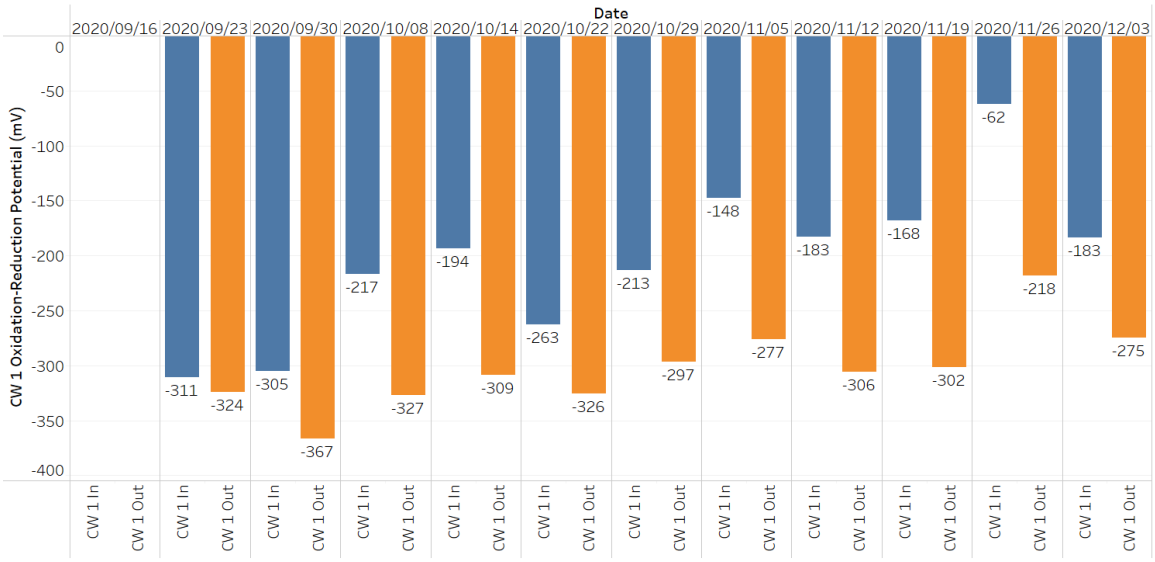

Figure 32: Results from oxidation-reduction potential (mV) tests conducted on influent and effluent streams of Constructed Wetland 1

Results from week 1 (16/09/2020) were not obtained as this was the first week visiting the CWs after the national COVID-19 lockdown in South Africa, and the entry point of CW 1 was not accessible with the ORP probe.

Table 18 shows the statistical summary of the results that are graphed in Figure 31.

Table 18: Statistical summary of results obtained from oxidation-reduction potential (mV) tests conducted on influent and effluent streams of Constructed Wetland 1

	ORP CW 1 In	ORP CW 1 Out
Mean (mV)	-204	-302
Min (mV)	-311.1	-366.6
Max (mV)	-62.3	-218.1
Range (mV)	248.8	148.5
Coefficient of Variation	-34.9	-12.5
p-value from paired t-test	0.000012	

As can been seen in Table 18, the p-value for this data is far lesser than 0.05 and there is therefore a statistically significant difference between inlet and outlet values of ORP in CW 1. These results are discussed further in Section 5.

4.1.6 pH

Figure 33 shows the pH of influent and effluent water in CW 1 over the 12-week study period.

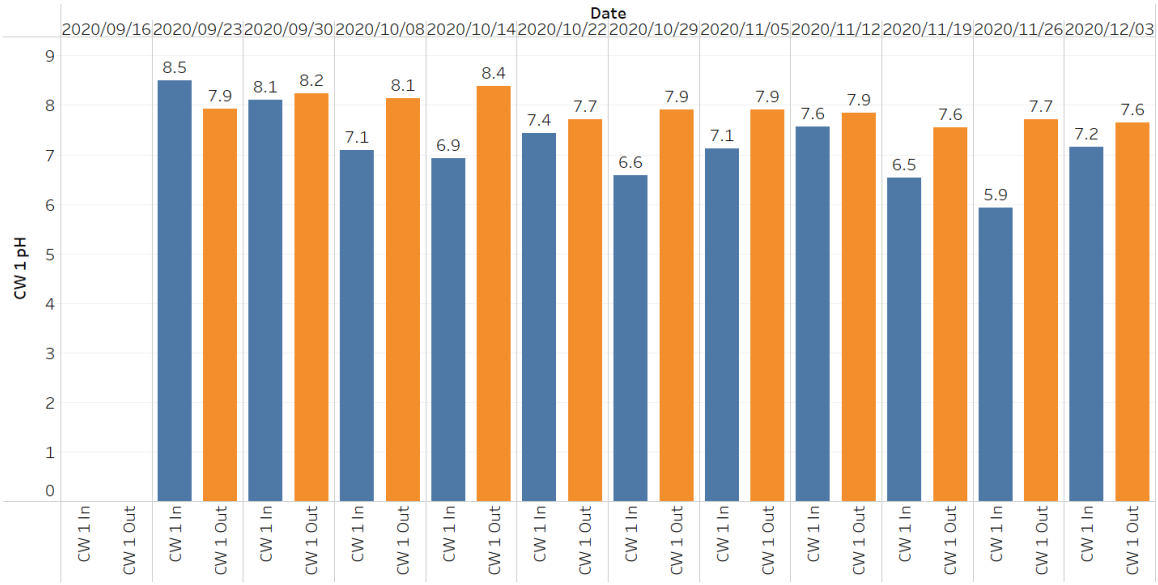

Figure 33: Results from pH tests conducted on influent and effluent streams of Constructed Wetland 1

Results from week 1 (16/09/2020) were not obtained as this was the first week visiting the CWs after the national COVID-19 lockdown in South Africa, and the entry point of CW 1 was not accessible with the pH probe.

Table 19 shows the statistical summary of the results that are graphed in Figure 33.

Table 19: Statistical summary of results obtained from pH tests conducted on influent and effluent streams of Constructed Wetland 1

	pH CW 1 In	pH CW 1 Out
Mean	7.2	7.9
Min	5.9	7.6
Max	8.5	8.4
Range	2.58	0.84
Coefficient of Variation	10.1	3.3
p-value from paired t-test	0.0054	

As can been seen in Table 19, the p-value for this data is far lesser than 0.05 and there is therefore a statistically significant difference between inlet and outlet values of pH in CW 1. These results are discussed further in Section 5.

4.1.7 Fluoride

Fluoride concentration was measured in influent and effluent water of CW 1 across the 12-week period of water sampling and analysis. Results from these tests are shown in Figure 34.

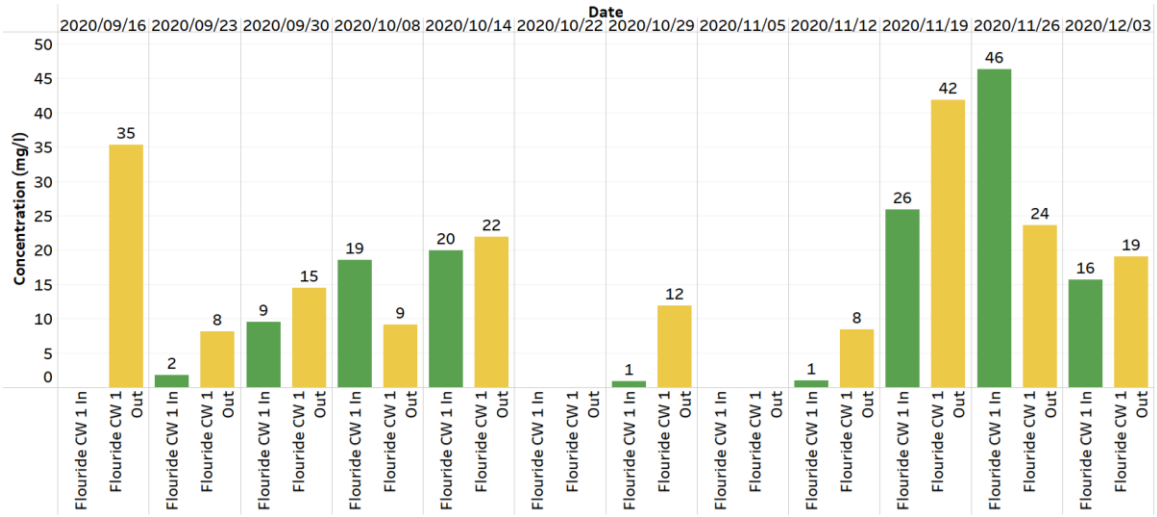

Figure 34: Results from fluoride concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 1

The fluoride concentration results for some weeks were beyond the detection limit of the ion chromatography machine used, and this is shown in Figure 34 by omitting the bar for that date.

Table 20 shows the statistical summary of the results that are graphed in Figure 34.

Table 20: Statistical summary of results obtained from fluoride concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 1

	Fluoride CW 1 In	Fluoride CW 1 Out
Mean (mg/l)	24	38
Min (mg/l)	1.0	8.2
Max (mg/l)	76.9	194.3
Range (mg/l)	75.9	186.2
Coefficient of Variation	100.6	157.3
p-value from paired t-test	0.33	

As can been seen from the p-value of 0.33 (which is greater than 0.05) for the data, there was not a statically significant difference between inlet and outlet values, and therefore these results cannot be analysed further to determine pollutant removal efficiency.

4.1.8 Chloride

Chloride concentration was measured in influent and effluent water of CW 1 across the 12-week period of water sampling and analysis. Results from these tests are shown in Figure 35.

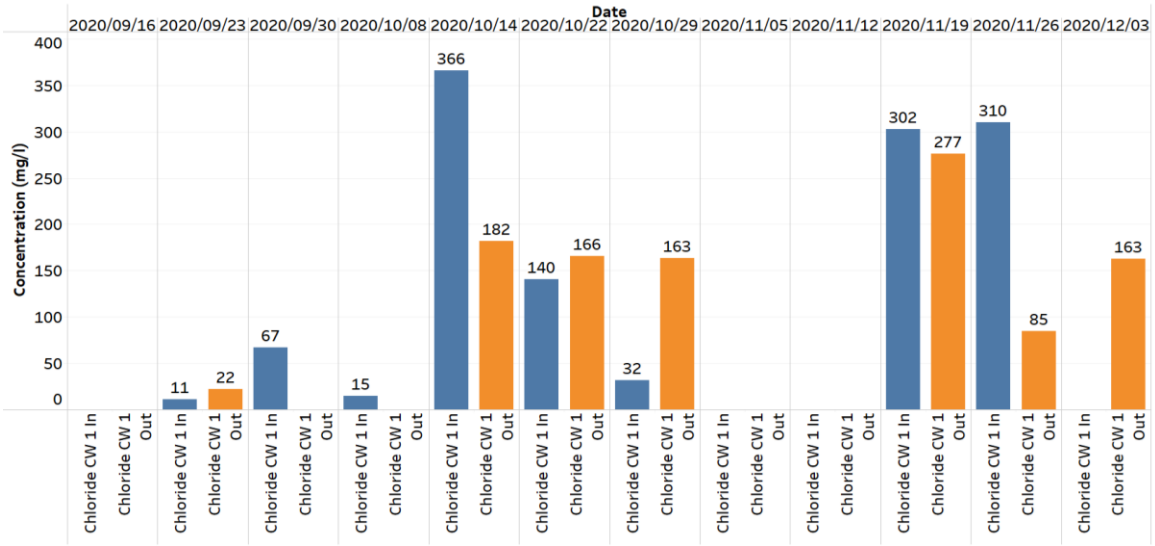

Figure 35:Results from chloride concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 1

The chloride concentration results for some weeks were beyond the detection limit of the ion chromatography machine used, and this is shown in Figure 35 by omitting the bar for that date.

Table 21 shows the statistical summary of the results that are graphed in Figure 35.

Table 21: Statistical summary of results obtained from chloride concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 1

	Chloride CW 1 In	Chloride CW 1 Out
Mean (mg/l)	194	149
Min (mg/l)	10.7	21.9
Max (mg/l)	366.1	276.5
Range (mg/l)	355.4	254.6
Coefficient of Variation	79.2	58.6
p-value from paired t-test	0.45	

The difference in the means has a p-value of 0.45 (which is greater than 0.05) which means there is no statistically significant difference between inlet and outlet values. Therefore, these are results are not analysed any further.

4.1.9 Nitrate

Nitrate concentration was measured in influent and effluent water of CW 1 across the 12-week period of water sampling and analysis. Results from these tests are shown in Figure 36.

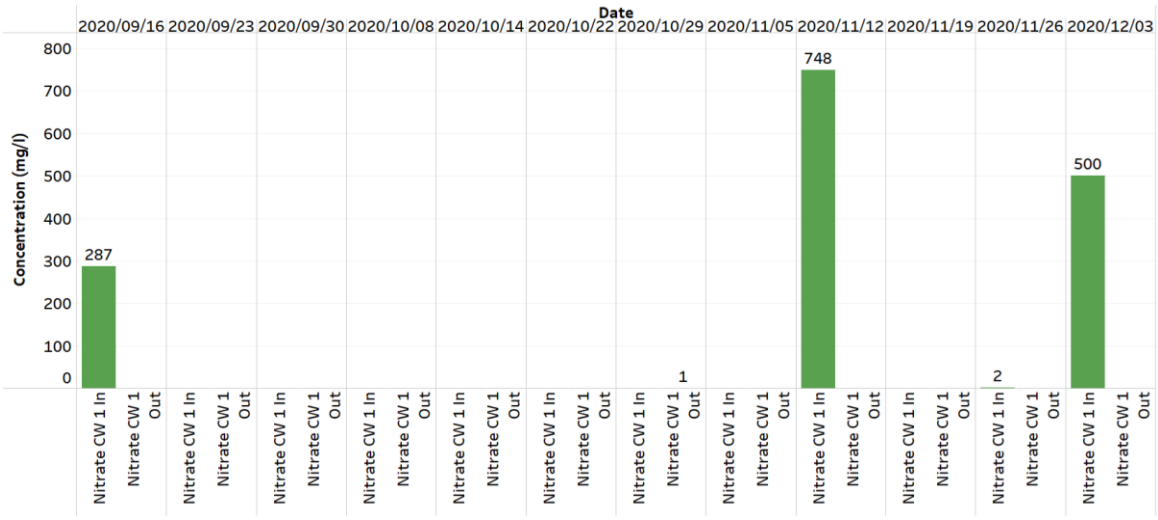

Figure 36: Results from nitrate concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 1

The nitrate concentration results for several weeks were beyond the detection limit of the ion chromatography machine used, and this is shown in Figure 36 by omitting the bar for that date. Although the researcher made every effort to obtain nitrate results by adjusting dilution concentrations for the sample placed in the ion chromatography machine and following the recommendations from the lab assistants, not many results were ultimately obtained for this ion. This can be attributed to the extremely high variability of water quality that was flowing through the wetlands. This is discussed in more detail in Section 5. Furthermore, because the results obtained did not contain at least two dates with measurements for both inlet and outlet vales, it was not possible to calculate a p-value for this dataset, and therefore the results are not statistically significant and cannot be used to calculate the pollutant removal efficiency for CW 1.

4.1.10 Nitrite

Nitrite concentration was measured in influent and effluent water of CW 1 across the 12-week period of water sampling and analysis. Results from these tests are shown in Figure 37.

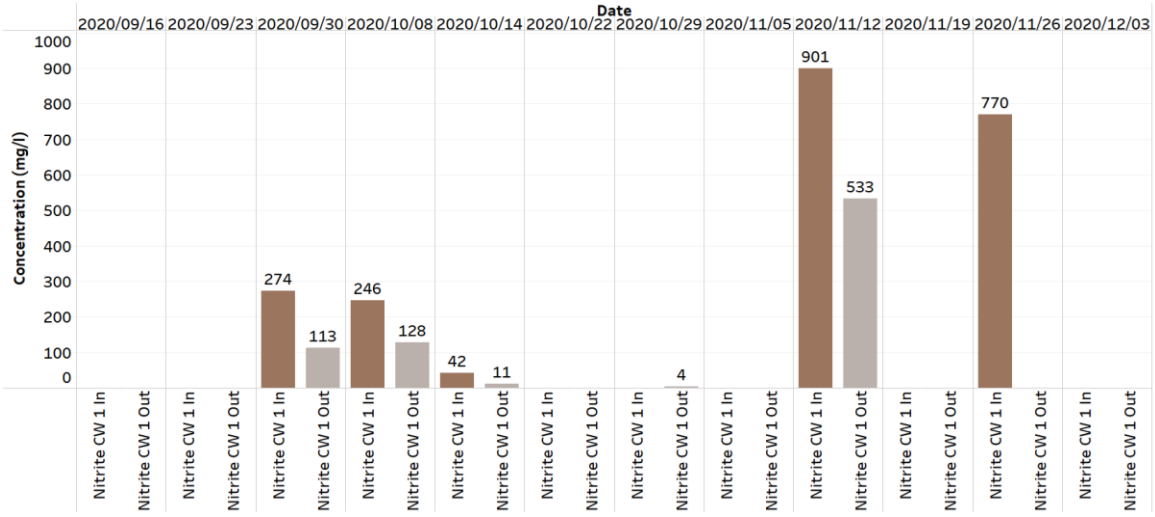

Figure 37: Results from nitrite concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 1

The nitrite concentration results for several weeks were beyond the detection limit of the ion chromatography machine used, and this is shown in Figure 37 by omitting the bar for that date. Although the researcher made every effort to obtain nitrite results by adjusting dilution concentrations for the sample placed in the ion chromatography machine and following the recommendations from the lab assistants, not many results were ultimately obtained for this ion. This can be attributed to the extremely high variability of water quality that was flowing through the wetlands. This is discussed in more detail in Section 5.

Table 22 shows the statistical summary of the results that are graphed in Figure 37.

Table 22: Statistical summary of results obtained from nitrite concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 1

	Nitrite CW 1 In	Nitrite CW 1 Out
Mean (mg/l)	366	196
Min (mg/l)	41.8	10.8
Max (mg/l)	900.7	532.5
Range (mg/l)	858.9	521.7
Coefficient of Variation	101.6	117.3
p-value from paired t-test	0.10	

The difference in the means has a p-value of 0.10 (which is greater than 0.05) which means there is no statistically significant difference between inlet and outlet values. Therefore, these are results are not analysed any further.

4.1.11 Phosphate

Phosphate concentration was measured in influent and effluent water of CW 1 across the 12-week period of water sampling and analysis. Results from these tests are shown in Figure 38.

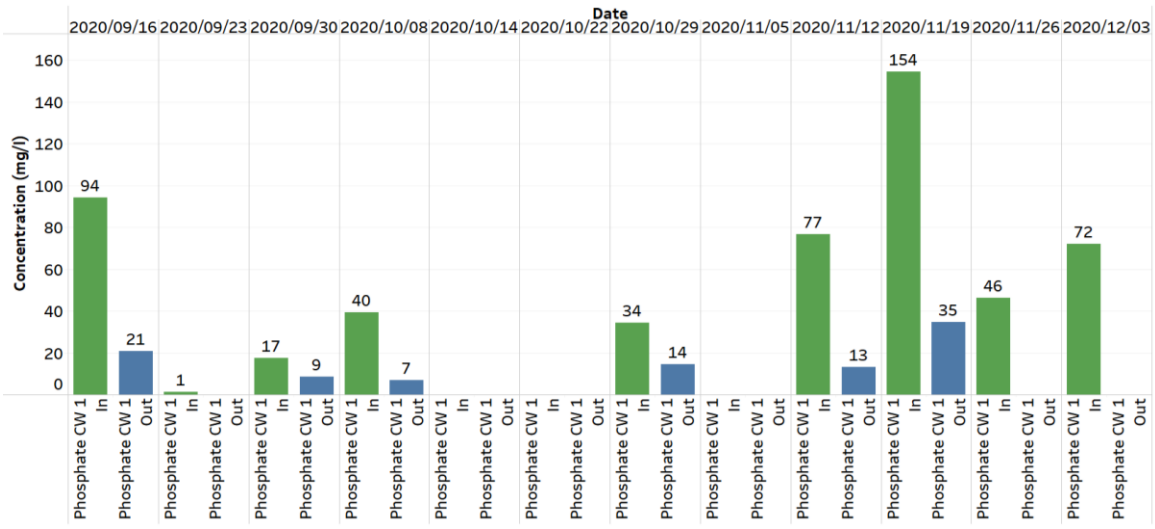

Figure 38: Results from phosphate concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 1

The phosphate concentration results for some weeks were beyond the detection limit of the ion chromatography machine used, and this is shown in Figure 38 by omitting the bar for that date.

Table 23 shows the statistical summary of the results that are graphed in Figure 38.

Table 23: Statistical summary of results obtained from phosphate concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 1

	Phosphate CW 1 In	Phosphate CW 1 Out
Mean (mg/l)	69	17
Min (mg/l)	17.4	7.0
Max (mg/l)	154.3	34.6
Range (mg/l)	136.9	27.5
Coefficient of Variation	72.6	61.1
p-value from paired t-test	0.03	

As can been seen in Table 23, the p-value for this data (0.03) is lesser than 0.05 and there is therefore a statistically significant difference between inlet and outlet values of phosphate concentration in CW 1. These results are discussed further in Section 5.

4.1.12 Sulfate

Sulfate concentration was measured in influent and effluent water of CW 1 across the 12-week period of water sampling and analysis. Results from these tests are shown in Figure 39.

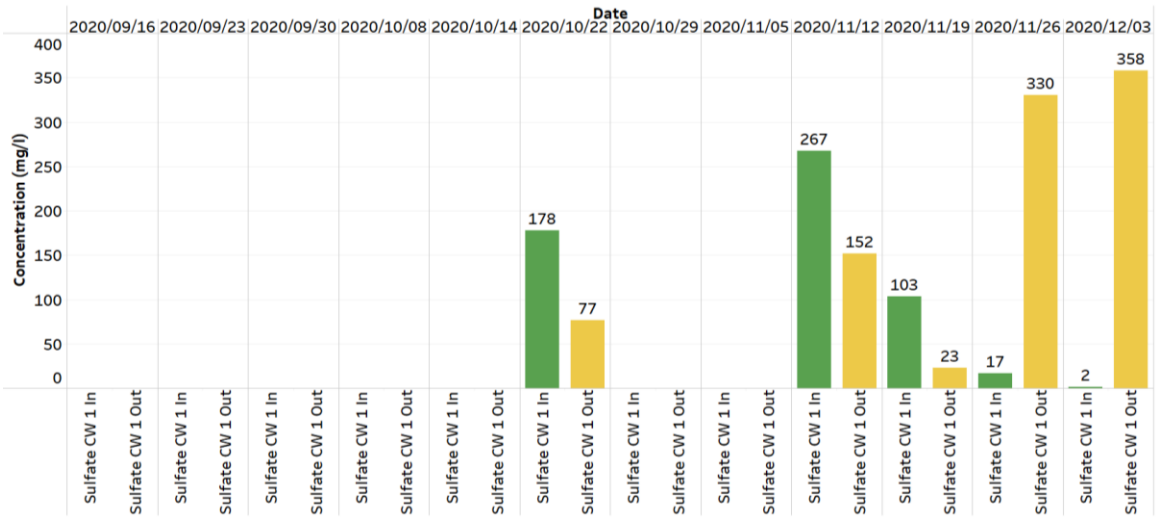

Figure 39: Results from sulfate concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 1

The sulfate concentration results for several weeks were beyond the detection limit of the ion chromatography machine used, and this is shown in Figure 39 by omitting the bar for that date. Although the researcher made every effort to obtain sulfate results by adjusting dilution concentrations for the sample placed in the ion chromatography machine and following the recommendations from the lab assistants, not many results were ultimately obtained for this ion. This can be attributed to the extremely high variability of water quality that was flowing through the wetlands. This is discussed in more detail in Section 5.

Table 24 shows the statistical summary of the results that are graphed in Figure 39.

Table 24: Statistical summary of results obtained from sulfate concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 1

	Sulfate CW 1 In	Sulfate CW 1 Out
Mean (mg/l)	114	188
Min (mg/l)	1.7	22.8
Max (mg/l)	267.4	357.8
Range (mg/l)	265.7	335.0
Coefficient of Variation	98.2	79.9
p-value from paired t-test	0.52	

The difference in the means has a p-value of 0.52 (which is greater than 0.05) which means there is no statistically significant difference between inlet and outlet values. Therefore, these are results are not analysed any further.

4.2 Constructed Wetland 2

This section explores the weekly results obtained from tests of chemical oxygen demand, *E. coli*, electrical conductivity, oxidation-reduction potential, pH and nutrient ions on influent and effluent water for CW 2. Further details regarding these results can be found in Appendix 1.

4.2.1 Chemical Oxygen Demand

Chemical oxygen demand of the influent and effluent water of CW 2 was measured weekly over the full 12-week period of water sampling and analysis. Results from these tests are shown in Figure 40.

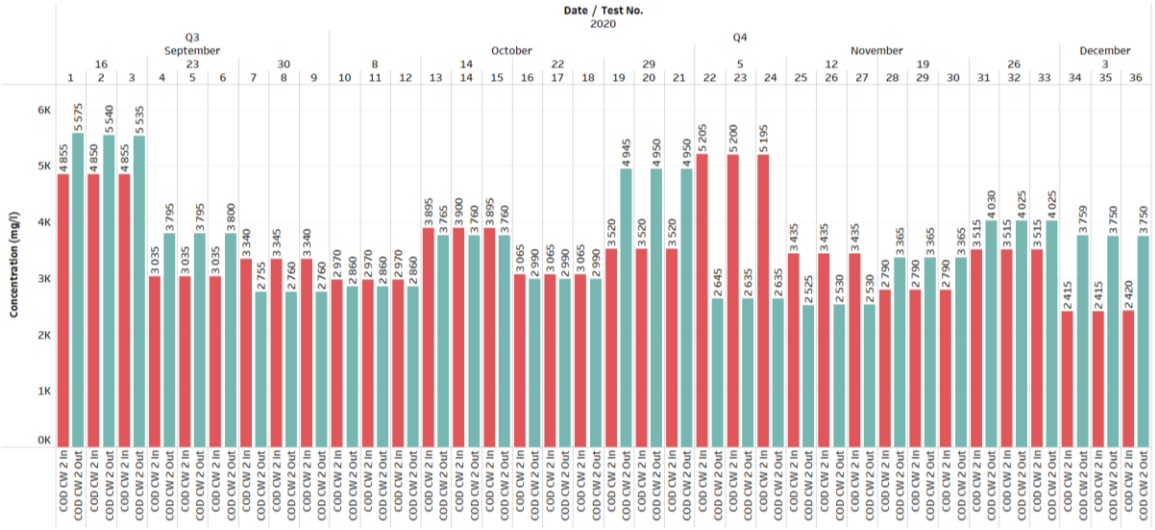

Figure 40: Results from chemical oxygen demand (COD) concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 2

Figure 40 shows results from each of the three spectrophotometer readings that were taken for each sample of water. Each reading number was given a unique test number ranging from 1 to 36 for both inlet and outlet sample readings.

Table 25 shows the statistical summary of the results that are graphed in Figure 40.

Table 25: Statistical summary of results obtained from chemical oxygen demand (COD) concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 2

	COD CW 2 In	COD CW 2 Out
Mean (mg/l)	3503	3580
Min (mg/l)	2415	2527

Max (mg/l)	5200	5540
Range (mg/l)	2785	3013
Coefficient of Variation	23.2	26.1
p-value from paired t-test	0.810	

The difference in the means has a p-value of 0.81 (which is greater than 0.05) which means there is no statistically significant difference between inlet and outlet values. Therefore, these are results are not analysed any further.

4.2.2 Total Coliforms

Total coliforms in the influent and effluent water of CW 2 were measured weekly over the full 12-week period of water sampling and analysis. Results from these tests are shown in Figure 41.

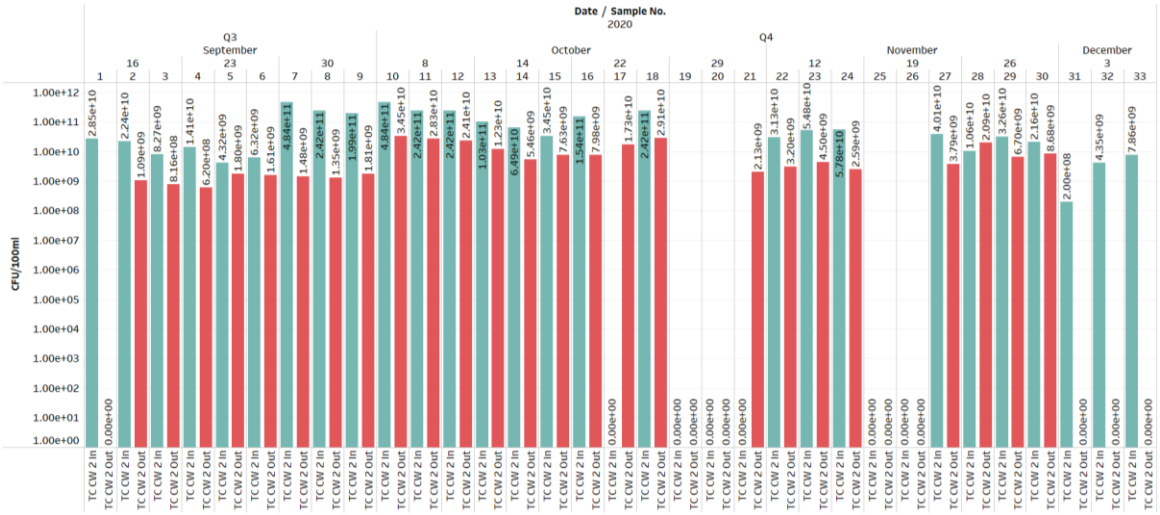

Figure 41: Results from total coliform (CFU/100ml) tests conducted on influent and effluent streams of Constructed Wetland 2

Figure 41 shows results from each of the triplicate samples that were tested for each week. Each of the triplicate samples for inlet and outlet streams was diluted to different concentrations to reduce the likelihood of samples being beyond detection limit after the incubation period, as discussed in Section 3.3.5. Each sample number was given a unique test number ranging from 1 to 33 for both inlet and outlet sample readings. Values that are shown as 0 on the graph were read as <1 CFU/100ml but are shown as 0 to differentiate between readings of <1 CFU/100ml and samples that were not taken. Readings from sampling week 8 (05 November 2020) were omitted as they were all unreadable due the colour change of the IDEXX kit cells not changing properly from clear to yellow. This

could be due to temperature change during the incubation period if someone tampered with the incubator during the 18–24-hour incubation period.

Table 26 shows the statistical summary of the results that are graphed in Figure 41.

Table 26: Statistical summary of results obtained from total coliform (CFU/100ml) tests conducted on influent and effluent streams of Constructed Wetland 2

	Total Coliforms CW 2 In	Total Coliforms CW 2 Out
Mean (CFU/100ml)	8.59E+10	6.96E+09
Min (CFU/100ml)	0	0
Max (CFU/100ml)	4.8392E+11	3.45E+10
Range (CFU/100ml)	4.8392E+11	3.45E+10
Coefficient of Variation	152.6	140.2
p-value from paired t-test	0.00106	

As can been seen in Table 26, the p-value for this data (0.00106) is far lesser than 0.05 and there is therefore a statistically significant difference between inlet and outlet concentrations of total coliforms in CW 2. Readings of <1 CFU/100ml (shown as 0 in Figure 41) were omitted from the p-value calculation. These results are discussed further in Section 5.

4.2.3 *Escherichia coli*

Escherichia coli in the influent and effluent water of CW 2 was measured weekly over the full 12-week period of water sampling and analysis. Results from these tests are shown in Figure 42.

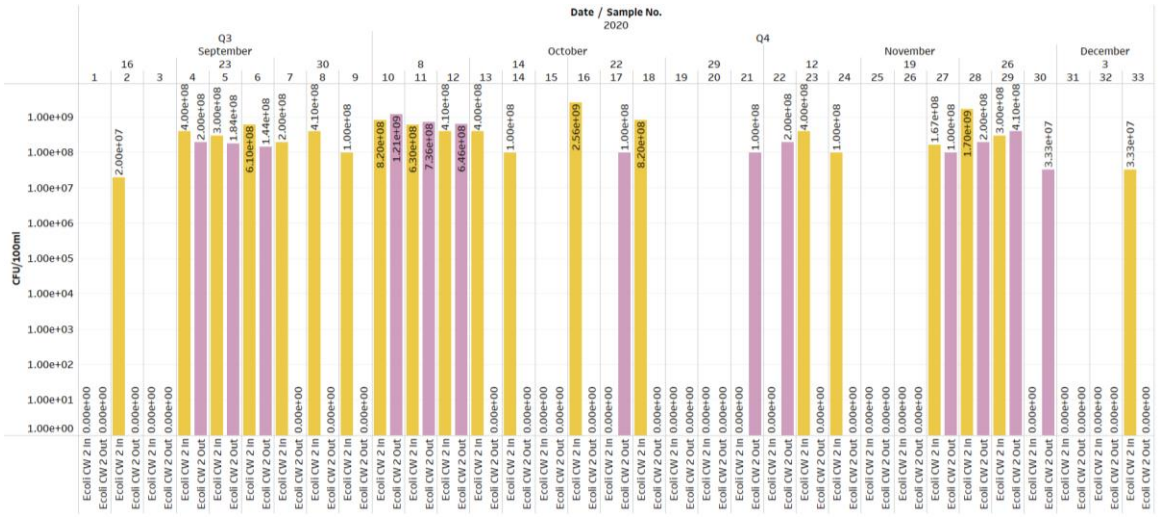

Figure 42: Results from *Escherichia coli* (CFU/100ml) tests conducted on influent and effluent streams of Constructed Wetland 2

Figure 42 shows results from each of the triplicate samples that were tested for each week. Each of the triplicate samples for inlet and outlet streams was diluted to different concentrations to reduce the likelihood of samples being beyond detection limit after the incubation period, as discussed in Section 3.3.5. Each sample number was given a unique test number ranging from 1 to 33 for both inlet and outlet sample readings. Readings from sampling week 8 (05 November 2020) were omitted as they were all unreadable due the colour change of the IDEXX kit cells not changing properly from clear to yellow. This could be due to temperature change during the incubation period if someone tampered with the incubator during the 18–24-hour incubation period.

Table 27 shows the statistical summary of the results that are graphed in Figure 42.

Table 27: Statistical summary of results obtained from *Escherichia coli* (CFU/100ml) tests conducted on influent and effluent streams of Constructed Wetland 2

	***E. coli* CW 2 In**	***E. coli* CW 2 Out**
Mean (CFU/100ml)	3.18E+08	1.29E+08
Min (CFU/100ml)	0	0
Max (CFU/100ml)	2.56E+09	1.21E+09
Range (CFU/100ml)	2.56E+09	1.21E+09
Coefficient of Variation	170.0	205.1
p-value from paired t-test	0.055	

The difference in the means has a p-value of 0.055 (which is greater than 0.05) which means there is no statistically significant difference between inlet and outlet values. Therefore, these are results are not analysed any further.

4.2.4 Electrical Conductivity

Electrical Conductivity (EC) for influent and effluent water of CW 2 was measured weekly over the 12-week period of water sampling and analysis, as shown in Figure 43.

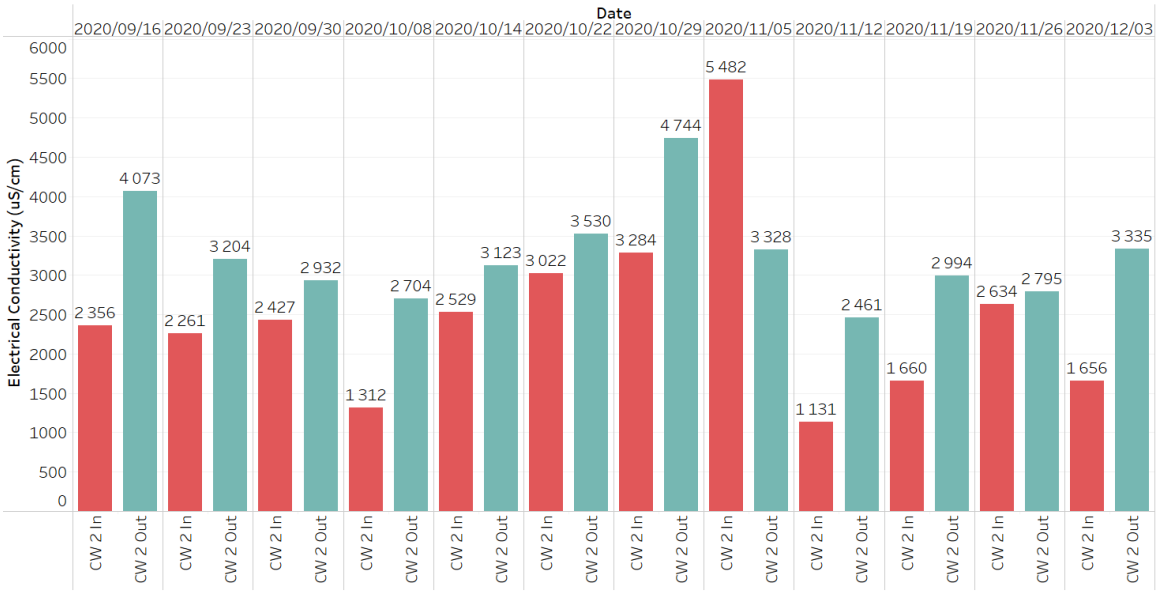

Figure 43: Results from electrical conductivity (µS/cm) tests conducted on influent and effluent streams of Constructed Wetland 2

As can be seen from the results in Figure 43, on average there is an increase in electrical conductivity from the inlet to the outlet of CW 2. This can likely be attributed to the hot climate in Alexandra which can lead to evapotranspiration and a net concentration of the water stream following through the CW. These results are discussed further in Section 5.

Table 28 shows the statistical summary of the results that are graphed in Figure 43.

Table 28: Statistical summary of results obtained from electrical conductivity (µS/cm) tests conducted on influent and effluent streams of Constructed Wetland 2

	EC CW 2 In	EC CW 2 Out
Mean (µS/cm)	2480	3269
Min (µS/cm)	1131	2461
Max (µS/cm)	5482	4744
Range (µS/cm)	4351	2283
Coefficient of Variation	46.4	19.1
p-value from paired t-test	0.026	

As can been seen in Table 28, the p-value (0.026) for this data is lesser than 0.05 and there is therefore a statistically significant difference between inlet and outlet values of pH in CW 1. These results are discussed further in Section 5.

4.2.5 Oxidation-Reduction Potential

Oxidation-reduction potential (ORP) was measured in influent and effluent water of CW 2 across the 12-week period of water sampling and analysis. Results from these tests are shown in Figure 44.

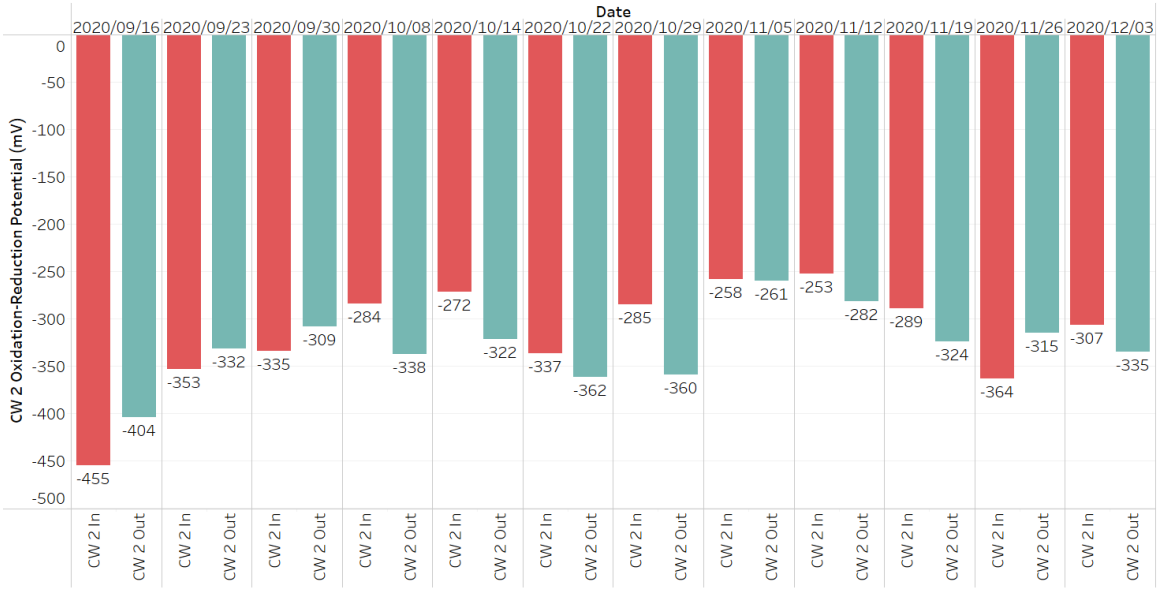

Figure 44: Results from oxidation-reduction potential (mV) tests conducted on influent and effluent streams of Constructed Wetland 2

Table 29 shows the statistical summary of the results that are graphed in Figure 44.

Table 29: Statistical summary of results obtained from oxidation-reduction potential (mV) tests conducted on influent and effluent streams of Constructed Wetland 2

	ORP CW 2 In	ORP CW 2 Out
Mean (mV)	-316	-329
Min (mV)	-455.1	-404.4
Max (mV)	-252.7	-260.5
Range (mV)	202.4	143.9
Coefficient of Variation	-18.1	-11.4
p-value from paired t-test	0.31	

The difference in the means has a p-value of 0.31 (which is greater than 0.05) which means there is no statistically significant difference between inlet and outlet values. Therefore, these are results are not analysed any further.

4.2.6 pH

Figure 45 shows the pH of influent and effluent water in CW 2 over the 12-week study period.



Figure 45: Results from pH tests conducted on influent and effluent streams of Constructed Wetland 2

Table 30 shows the statistical summary of the results that are graphed in Figure 45.

Table 30: Statistical summary of results obtained from pH tests conducted on influent and effluent streams of Constructed Wetland 2

	pH CW 2 In	pH CW 2 Out
Mean	8	8
Min	7.66	7.84
Max	9.16	8.8
Range	1.5	0.96
Coefficient of Variation	6.0	3.6
p-value from paired t-test	0.36	

The difference in the means has a p-value of 0.36 (which is greater than 0.05) which means there is no statistically significant difference between inlet and outlet values. Therefore, these are results are not analysed any further.

4.2.7 Fluoride

Fluoride concentration was measured in influent and effluent water of CW 2 across the 12-week period of water sampling and analysis. Results from these tests are shown in Figure 46.

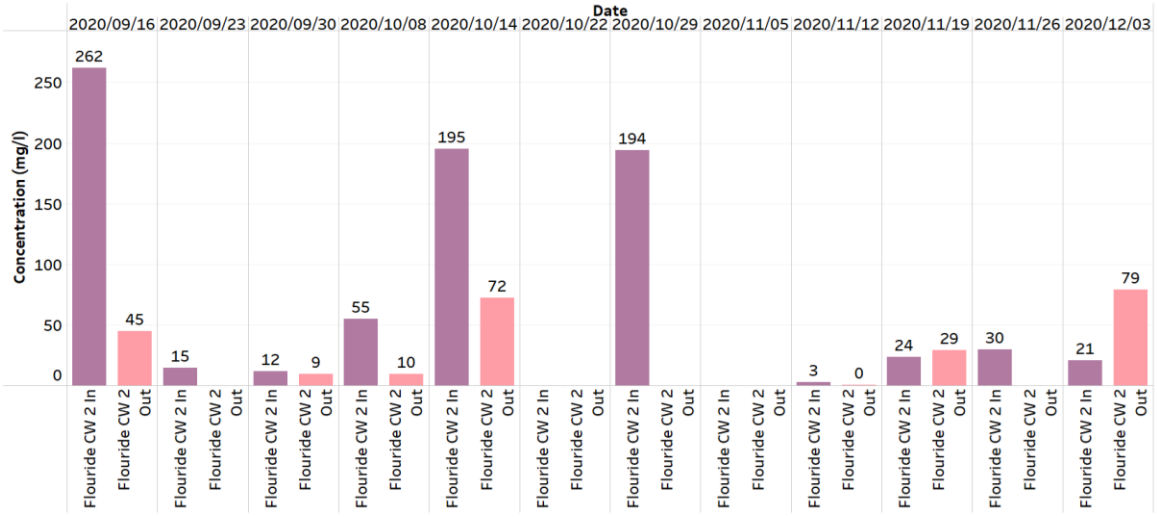

Figure 46: Results from fluoride concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 2

The fluoride concentration results for some weeks were beyond the detection limit of the ion chromatography machine used, and this is shown in Figure 46 by omitting the bar for that date.

Table 31 shows the statistical summary of the results that are graphed in Figure 46.

Table 31: Statistical summary of results obtained from fluoride concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 2

	Fluoride CW 2 In	Fluoride CW 2 Out
Mean (mg/l)	82	35
Min (mg/l)	3.0	0.5
Max (mg/l)	261.8	79.3
Range (mg/l)	258.9	78.8
Coefficient of Variation	126.7	90.2
p-value from paired t-test	0.23	

The difference in the means has a p-value of 0.23 (which is greater than 0.05) which means there is no statistically significant difference between inlet and outlet values. Therefore, these are results are not analysed any further.

4.2.8 Chloride

Chloride concentration was measured in influent and effluent water of CW 2 across the 12-week period of water sampling and analysis. Results from these tests are shown in Figure 47.

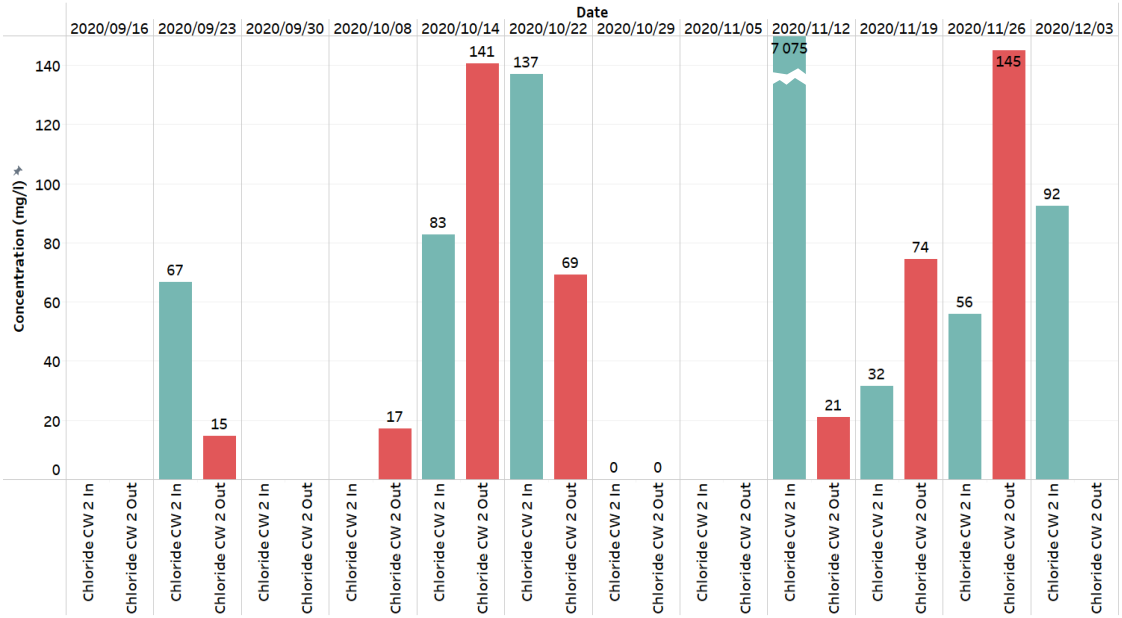

Figure 47: Results from chloride concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 2

The chloride concentration results for some weeks were beyond the detection limit of the ion chromatography machine used, and this is shown in Figure 35 by omitting the bar for that date. Furthermore, a very clear outlier was obtained in week 9 of sampling (12/11/2020) and the data points for inlet and outlet concentration for this week were omitted from the calculation of the p-value for the data set. This outlier is confirmed statistically by being more than 2 standard deviations from the mean.

Table 32 shows the statistical summary of the results that are graphed in Figure 47.

Table 32: Statistical summary of results obtained from chloride concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 2

	Chloride CW 2 In	Chloride CW 2 Out
Mean (mg/l)	75	89
Min (mg/l)	31.7	14.7
Max (mg/l)	136.9	145.0
Range (mg/l)	105.2	130.3
Coefficient of Variation	52.7	61.5
p-value from paired t-test	0.68	

The difference in the means has a p-value of 0.68 (which is greater than 0.05) which means there is no statistically significant difference between inlet and outlet values. Therefore, these are results are not analysed any further.

4.2.9 Nitrate

Nitrate concentration was measured in influent and effluent water of CW 2 across the 12-week period of water sampling and analysis. Results from these tests are shown in Figure 48.

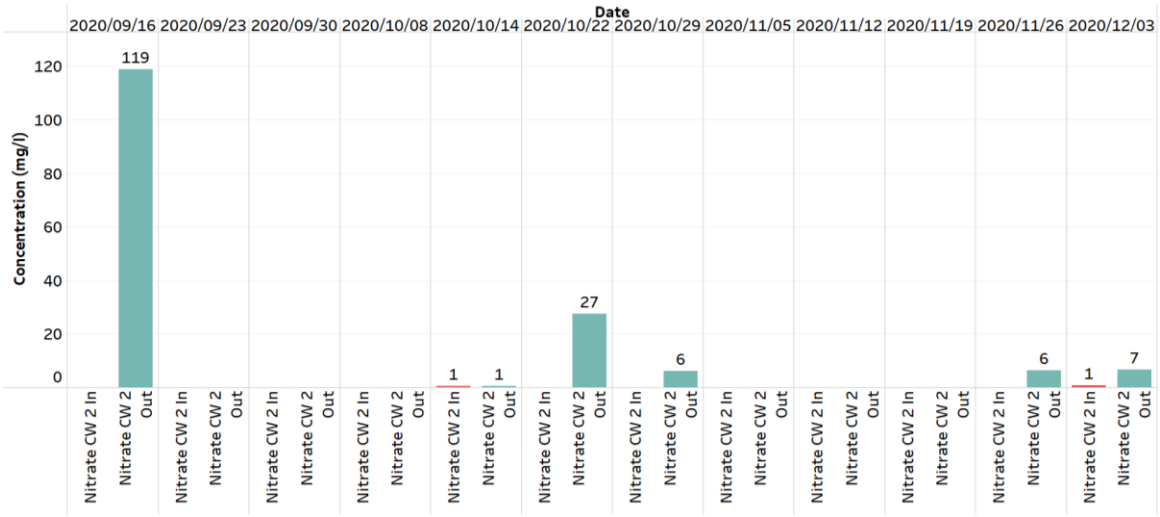

Figure 48: Results from nitrate concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 2

The nitrate concentration results for several weeks were beyond the detection limit of the ion chromatography machine used, and this is shown in Figure 48 by omitting the bar for that date. Although the researcher made every effort to obtain nitrate results by adjusting dilution concentrations for the sample placed in the ion chromatography machine and following the recommendations from the lab assistants, not many results were ultimately obtained for this ion. This can be attributed to the extremely high variability of water quality that was flowing through the wetlands. This is discussed in more detail in Section 5.

Table 33 shows the statistical summary of the results that are graphed in Figure 48.

Table 33: Statistical summary of results obtained from nitrate concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 2

	Nitrate CW 2 In	Nitrate CW 2 Out
Mean (mg/l)	1	4
Min (mg/l)	0.5	0.6
Max (mg/l)	0.9	6.7
Range (mg/l)	0.3	6.1
Coefficient of Variation	31.7	117.6
p-value from paired t-test	0.49	

The difference in the means has a p-value of 0.49 (which is greater than 0.05) which means there is no statistically significant difference between inlet and outlet values. Therefore, these are results are not analysed any further.

4.2.10 Nitrite

Nitrite concentration was measured in influent and effluent water of CW 2 across the 12-week period of water sampling and analysis. Results from these tests are shown in Figure 49.

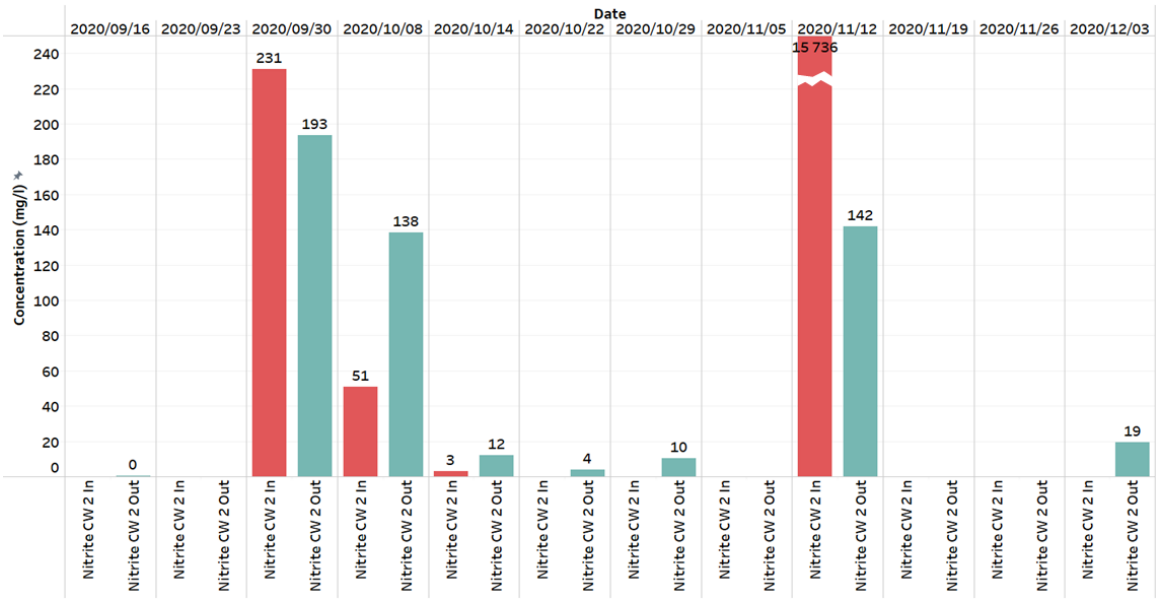

Figure 49: Results from nitrite concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 2

The nitrite concentration results for several weeks were beyond the detection limit of the ion chromatography machine used, and this is shown in Figure 49 by omitting the bar for that date. Although the researcher made every effort to obtain nitrite results by adjusting dilution concentrations for the sample placed in the ion chromatography machine and following the recommendations from the lab assistants, not many results were ultimately obtained for this ion. This can be attributed to the extremely high variability of water quality that was flowing through the wetlands. This is discussed in more detail in Section 5. Furthermore, a very clear outlier was obtained in week 9 of sampling (12/11/2020) and the data points for inlet and outlet concentration for this week were omitted from the calculation of the p-value for the data set. This outlier is confirmed statistically by being more than 2 standard deviations from the mean.

Table 34 shows the statistical summary of the results that are graphed in Figure 49.

Table 34: Statistical summary of results obtained from nitrite concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 2

	Nitrite CW 2 In	Nitrite CW 2 Out
Mean (mg/l)	95	114
Min (mg/l)	3.1	11.9
Max (mg/l)	230.9	193.3
Range (mg/l)	227.7	181.4
Coefficient of Variation	126.5	81.3
p-value from paired t-test	0.65	

The difference in the means has a p-value of 0.65 (which is greater than 0.05) which means there is no statistically significant difference between inlet and outlet values. Therefore, these are results are not analysed any further.

4.2.11 Phosphate

Phosphate concentration was measured in influent and effluent water of CW 2 across the 12-week period of water sampling and analysis. Results from these tests are shown in Figure 50.

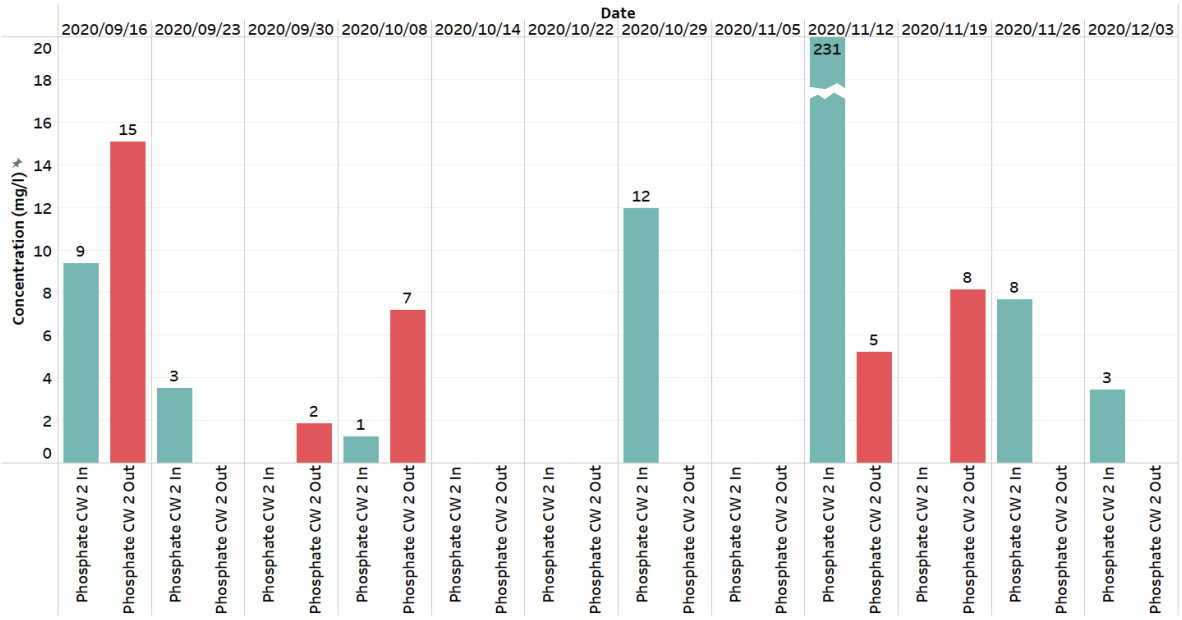

Figure 50: Results from phosphate concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 2

The phosphate concentration results for some weeks were beyond the detection limit of the ion chromatography machine used, and this is shown in Figure 50 by omitting the bar for that date. Furthermore, a very clear outlier was obtained in week 9 of sampling (12/11/2020) and the data points for inlet and outlet concentration for this week were

omitted from the calculation of the p-value for the data set. This outlier is confirmed statistically by being more than 2 standard deviations from the mean.

Table 35 shows the statistical summary of the results that are graphed in Figure 50.

Table 35: Statistical summary of results obtained from phosphate concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 2

	Phosphate CW 2 In	Phosphate CW 2 Out
Mean (mg/l)	5	11
Min (mg/l)	1.3	7.2
Max (mg/l)	9.3	15.1
Range (mg/l)	8.1	7.9
Coefficient of Variation	108.0	50.2
p-value from paired t-test	0.01	

As can been seen in Table 23, the p-value for this data (0.03) is lesser than 0.05 and there is therefore a statistically significant difference between inlet and outlet values of phosphate concentration in CW 1. These results are discussed further in Section 5.

4.2.12 Sulfate

Sulfate concentration was measured in influent and effluent water of CW 2 across the 12-week period of water sampling and analysis. Results from these tests are shown in Figure 51.

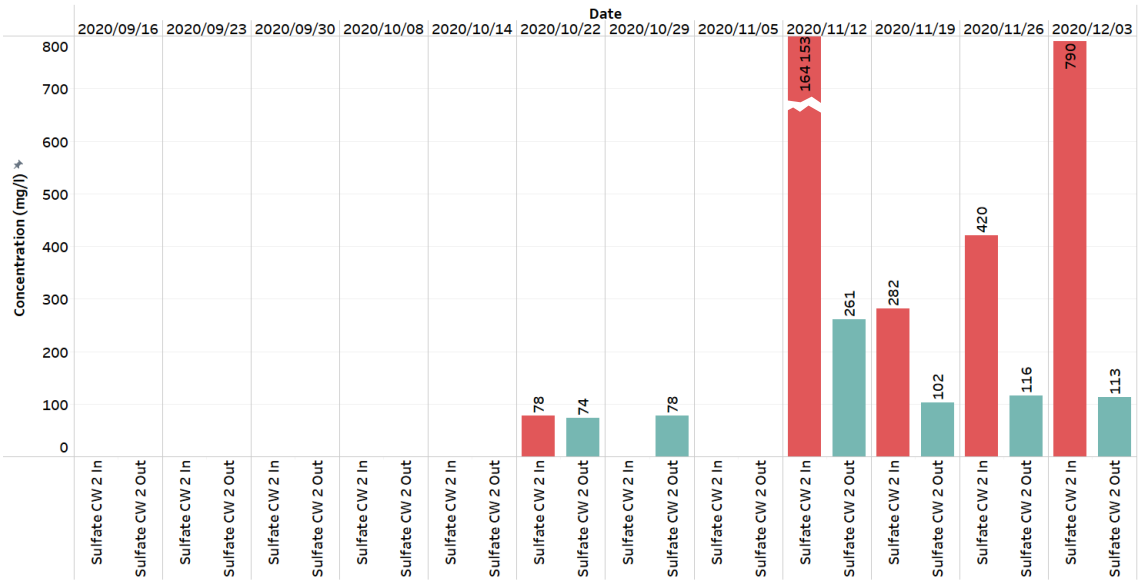

Figure 51: Results from sulfate concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 2

The sulfate concentration results for several weeks were beyond the detection limit of the ion chromatography machine used, and this is shown in Figure 51 by omitting the bar for that date. Although the researcher made every effort to obtain sulfate results by adjusting dilution concentrations for the sample placed in the ion chromatography machine and following the recommendations from the lab assistants, not many results were ultimately obtained for this ion. This can be attributed to the extremely high variability of water quality that was flowing through the wetlands. This is discussed in more detail in Section 5. Furthermore, a very clear outlier was obtained in week 9 of sampling (12/11/2020) and the data points for inlet and outlet concentration for this week were omitted from the calculation of the p-value for the data set. This outlier is confirmed statistically by being more than 2 standard deviations from the mean.

Table 36 shows the statistical summary of the results that are graphed in Figure 51.

Table 36: Statistical summary of results obtained from sulfate concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 2

	Sulfate CW 2 In	**Sulfate CW 2 Out**
Mean (mg/l)	392	101
Min (mg/l)	78.3	73.7
Max (mg/l)	789.6	116.0
Range (mg/l)	711.4	42.4
Coefficient of Variation	76.4	19.1
p-value from paired t-test	0.13	

The difference in the means has a p-value of 0.13 (which is greater than 0.05) which means there is no statistically significant difference between inlet and outlet values. Therefore, these are results are not analysed any further.

4.3 Jukskei River

This section explored the weekly results obtained from tests of chemical oxygen demand, *E. coli*, electrical conductivity, oxidation-reduction potential and pH on samples from the Jukskei River upstream and downstream of Alexandra. It is important to note that the gaps in the data collected are mostly due to security concerns with collecting samples from the Jukskei River. The researcher made his best effort to collect as many samples from the river as feasible, but the security concerns in some weeks only allowed for downstream samples to be collected. Further details regarding these results and those from other quality parameters can be found in Appendix 1.

4.3.1 Chemical Oxygen Demand

Figure 52 shows the results from tests measuring the chemical oxygen demand in samples collected from the Jukskei River, upstream and downstream of Alexandra.

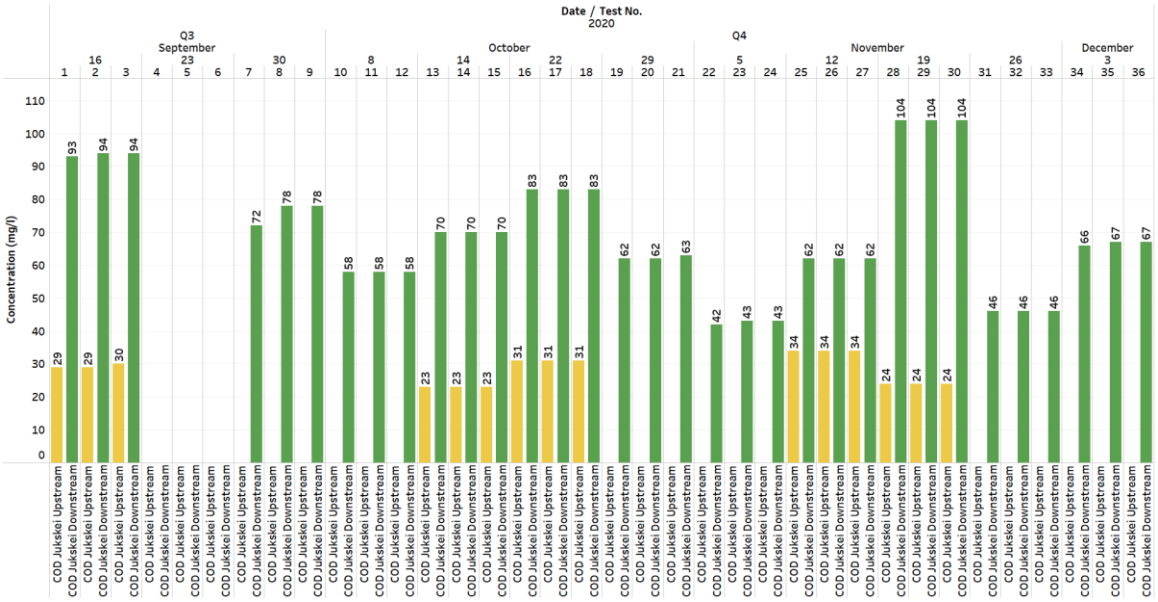

Figure 52: Results from chemical oxygen demand (COD) concentration (mg/l) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

Figure 52 shows results from each of the three spectrophotometer readings that were taken for each sample of water. Each reading number was given a unique test number ranging from 1 to 36 for both inlet and outlet sample readings. The gaps in data for upstream samples were all due to samples not being taken for security concerns, and the gap in data for downstream samples in week 2 (23 September 2020) is due to the results being beyond the detection limit of the reagents used to process the sample.

Table 37 shows the statistical summary of the results that are graphed in Figure 52.

Table 37: Statistical summary of results obtained from chemical oxygen demand (COD) concentration (mg/l) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

	COD Jukskei Upstream	COD Jukskei Downstream
Mean (mg/l)	28	83
Min (mg/l)	23	62
Max (mg/l)	34	104
Range (mg/l)	11	42
Coefficient of Variation	15.3	19.1
p-value from paired t-test	0.0000000133	

As can been seen in Table 26, the p-value for this data is far lesser than 0.05 and there is therefore a statistically significant difference between upstream and downstream concentrations of COD in the Jukskei River. These results are discussed further in Section 5.

4.3.2 Total Coliforms

Figure 53 shows the results from tests measuring the total coliforms in samples collected from the Jukskei River, upstream and downstream of Alexandra.

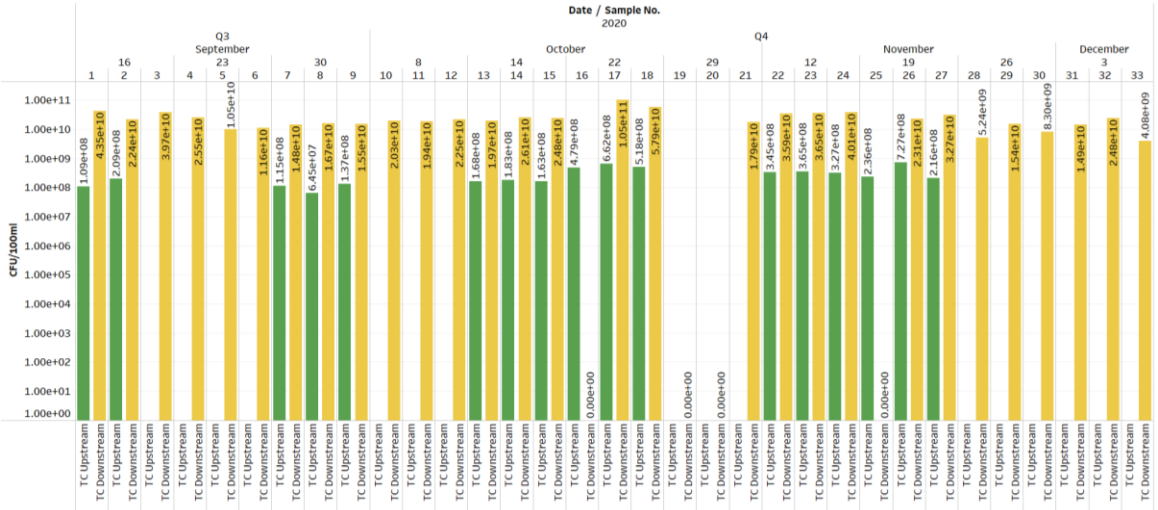

Figure 53: Results from total coliform (CFU/100ml) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

Figure 53 shows results from each of the triplicate samples that were tested for each week. Each of the triplicate samples for inlet and outlet streams was diluted to different

concentrations to reduce the likelihood of samples being beyond detection limit after the incubation period, as discussed in Section 3.3.5. Each sample number was given a unique test number ranging from 1 to 33 for both inlet and outlet sample readings. Values that are shown as 0 on the graph were read as <1 CFU/100ml but are shown as 0 to differentiate between readings of <1 CFU/100ml and samples that were not taken. Readings from sampling week 8 (05 November 2020) were omitted as they were all unreadable due the colour change of the IDEXX kit cells not changing properly from clear to yellow. This could be due to temperature change during the incubation period if someone tampered with the incubator during the 18–24-hour incubation period.

Table 38 shows the statistical summary of the results that are graphed in Figure 53.

Table 38: Statistical summary of results obtained from total coliform (CFU/100ml) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

	Total Coliforms Jukskei Upstream	**Total Coliforms Jukskei Downstream**
Mean (CFU/100ml)	2.95E+08	3.03E+10
Min (CFU/100ml)	6.45E+07	0
Max (CFU/100ml)	7.27E+08	1.0462E+11
Range (CFU/100ml)	6.63E+08	1.0462E+11
Coefficient of Variation	66.7	80.0
p-value from paired t-test	0.000103	

As can been seen in Table 38, the p-value for this data (0.000103) is far lesser than 0.05 and there is therefore a statistically significant difference between upstream and downstream concentrations of total coliforms in the Jukskei River. Readings of <1 CFU/100ml (shown as 0 in Figure 53) were omitted from the p-value calculation. These results are discussed further in Section 5.

4.3.3 *Escherichia coli*

Figure 54 shows the concentration of *E. coli* colony forming units in samples collected from the Jukskei River, upstream and downstream of Alexandra. It is important to note that this graph has been plotted on a logarithmic scale due to the large difference in concentration between upstream and downstream samples.

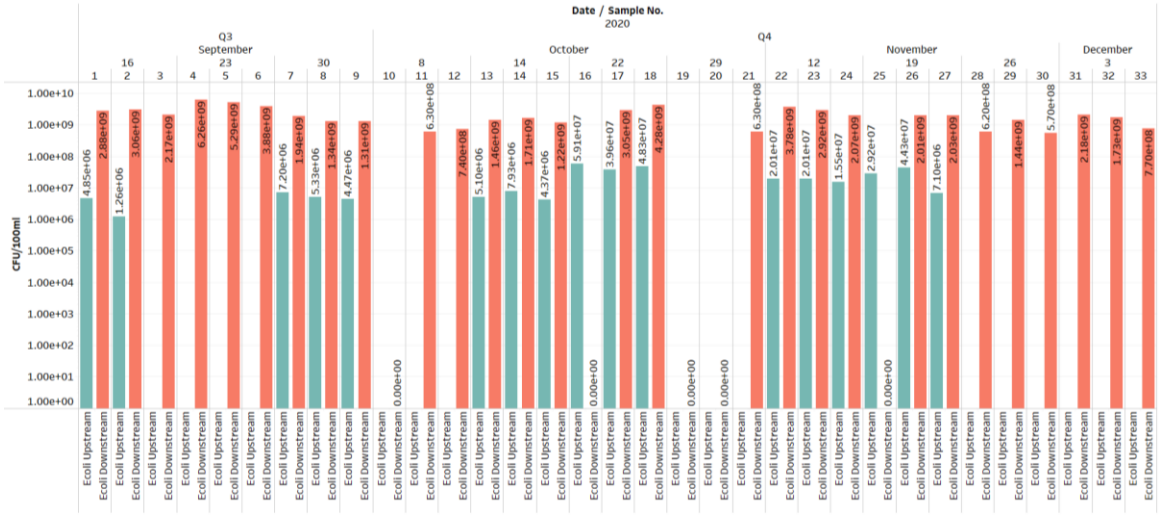

Figure 54: Results from *Escherichia coli* (CFU/100ml) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

Figure 54 shows results from each of the triplicate samples that were tested for each week. Each of the triplicate samples for inlet and outlet streams was diluted to different concentrations to reduce the likelihood of samples being beyond detection limit after the incubation period, as discussed in Section 3.3.5. Each sample number was given a unique test number ranging from 1 to 33 for both inlet and outlet sample readings. Readings from sampling week 8 (05 November 2020) were omitted as they were all unreadable due the colour change of the IDEXX kit cells not changing properly from clear to yellow. This could be due to temperature change during the incubation period if someone tampered with the incubator during the 18–24-hour incubation period.

Table 39 shows the statistical summary of the results that are graphed in Figure 54.

Table 39: Statistical summary of results obtained from *Escherichia coli* (CFU/100ml) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

	***E. coli* Jukskei Upstream**	***E. coli* Jukskei Downstream**
Mean (CFU/100ml)	1.90E+07	2.06E+09
Min (CFU/100ml)	1.26E+06	0
Max (CFU/100ml)	5.91E+07	4.28E+09
Range (CFU/100ml)	5.78E+07	4.28E+09
Coefficient of Variation	96.2	56.9
p-value from paired t-test	0.00000221	

As can been seen in Table 39, the p-value for this data (0.00000221) is far lesser than 0.05 and there is therefore a statistically significant difference between upstream and downstream concentrations of E. coli in the Jukskei River. Readings of <1 CFU/100ml

(shown as 0 in Figure 54) were omitted from the p-value calculation. These results are discussed further in Section 5.

4.3.4 Electrical Conductivity

Figure 55 shows the results from tests measuring electrical conductivity in samples from the Jukskei River, upstream and downstream of Alexandra.

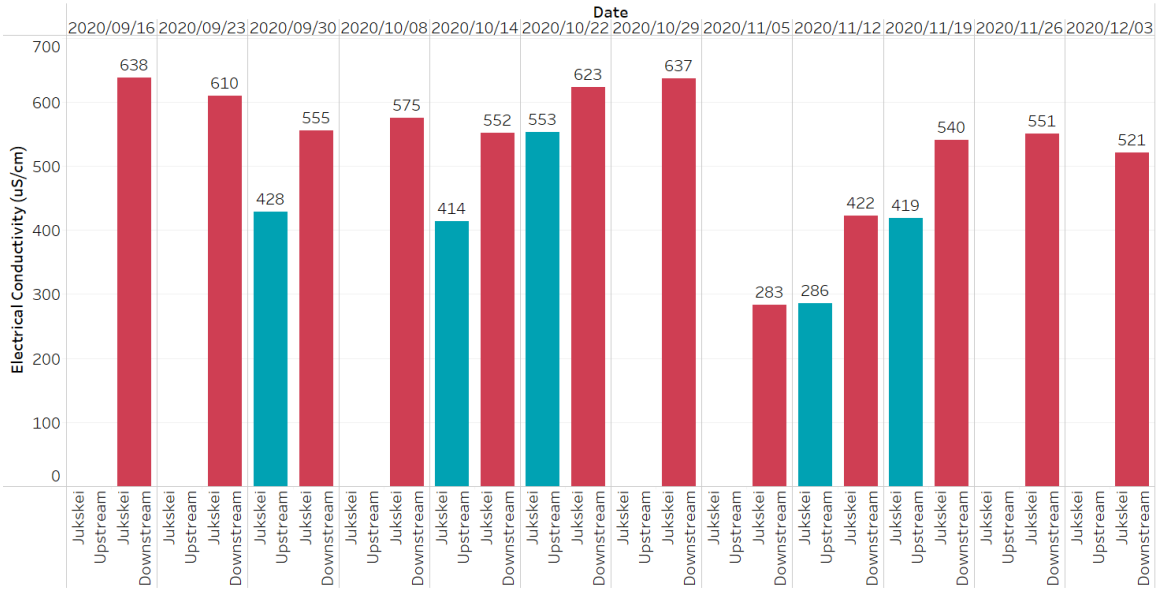

Figure 55: Results from electrical conductivity (µS/cm) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

As can be seen from the results in Figure 55, on average there is an increase in electrical conductivity between the upstream to the downstream points on the Jukskei River. This can likely be attributed to the anthropogenic pollution that enters the Jukskei River from Alexandra which. These results are discussed further in Section 5.

Table 40 shows the statistical summary of the results that are graphed in Figure 55.

Table 40: Statistical summary of results obtained from electrical conductivity (µS/cm) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

	EC Jukskei Upstream	EC Jukskei Downstream
Mean (µS/cm)	420	538
Min (µS/cm)	286	422
Max (µS/cm)	553	623
Range (µS/cm)	267	201
Coefficient of Variation	22.5	13.5
p-value from paired t-test	0.000690	

As can been seen in Table 40, the p-value (0.000690) for this data is lesser than 0.05 and there is therefore a statistically significant difference between upstream and downstream values of electrical conductivity in the Jukskei River. These results are discussed further in Section 5.

4.3.5 Oxidation-Reduction Potential

Oxidation-reduction potential measured in the Jukskei River upstream and downstream of Alexandra is shown in Figure 56.



Figure 56: Results from oxidation-reduction potential (mV) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

The results indicate that the pollution entering the Jukskei River has a net reductive effect on the river water, as can be seen my most downstream river samples having a negative ORP. These results are discussed further in Section 5.

Table 41 shows the statistical summary of the results that are graphed in Figure 56.

Table 41: Statistical summary of results obtained from oxidation-reduction potential (mV) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

	ORP Jukskei Upstream	ORP Jukskei Downstream
Mean (mV)	61	-45
Min (mV)	-41.7	-122.1
Max (mV)	200.1	28.9
Range (mV)	241.8	151
Coefficient of Variation	NA	NA
p-value from paired t-test	0.0439	

As can been seen in Table 41, the p-value (0.0439) for this data is lesser than 0.05 and there is therefore a statistically significant difference between upstream and downstream values of oxidation-reduction potential in the Jukskei River. These results are discussed further in Section 5.

4.3.6 pH

The pH of samples collected from the Jukskei River, upstream and downstream of Alexandra are plotted in Figure 57.

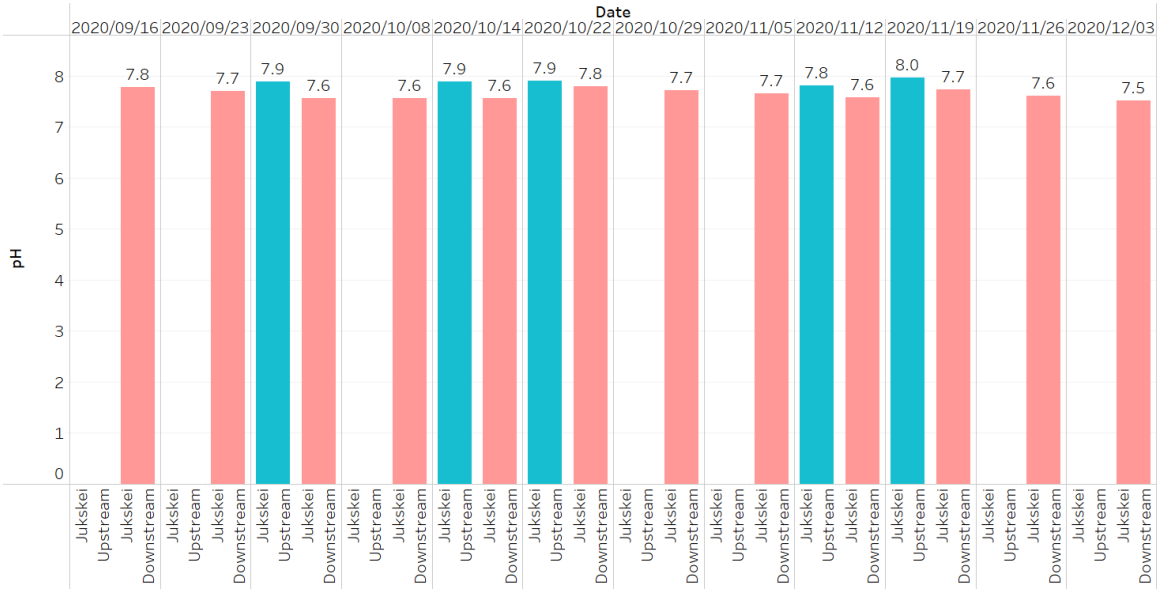

Figure 57: Results from pH tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

In general, it can be seen that pH values are stable for both upstream and downstream samples, with little variation. The pollution that enters the Jukskei river has a net alkalinising effect on the water, as the downstream pH is generally has a higher pH that upstream results. These results are discussed further in Section 5.

Table 42 shows the statistical summary of the results that are graphed in Figure 57.

Table 42: Statistical summary of results obtained from pH tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

	pH Jukskei Upstream	pH Jukskei Downstream
Mean	7.89	7.65
Min	7.81	7.56
Max	7.97	7.8
Range	0.16	0.24
Coefficient of Variation	0.7	1.5
p-value from paired t-test	0.00355	

As can been seen in Table 42, the p-value (0.00355) for this data is lesser than 0.05 and there is therefore a statistically significant difference between upstream and downstream values of pH in the Jukskei River. These results are discussed further in Section 5.

4.3.7 Fluoride

Figure 58 shows the results from tests measuring fluoride concentration in samples from the Jukskei River, upstream and downstream of Alexandra.

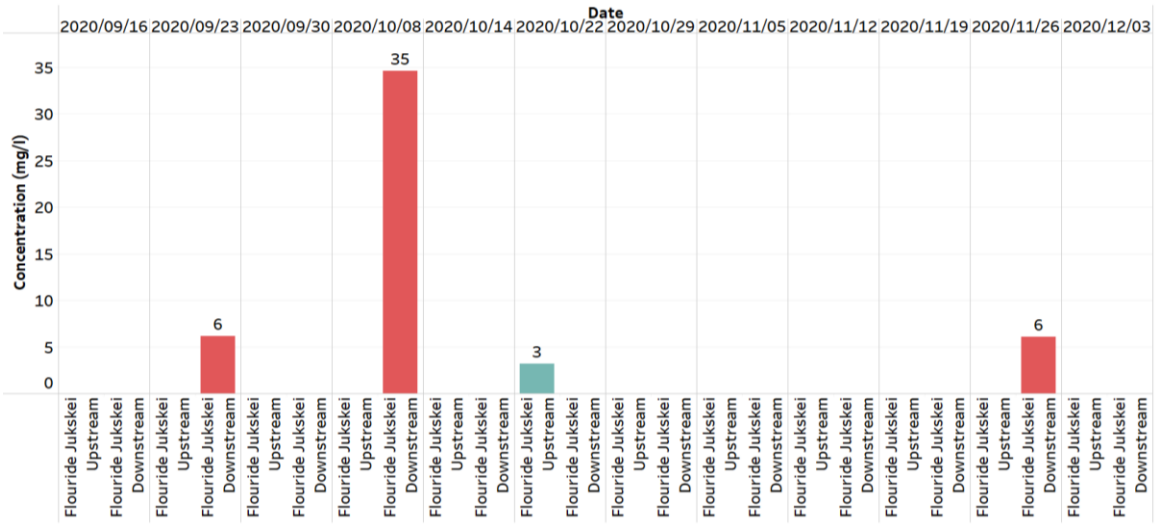

Figure 58: Results from fluoride concentration (mg/l) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

The fluoride concentration results for several weeks were beyond the detection limit of the ion chromatography machine used, and this is shown in Figure 58 by omitting the bar for that date. Although the researcher made every effort to obtain fluoride results by adjusting dilution concentrations for the sample placed in the ion chromatography machine and following the recommendations from the lab assistants, not many results were ultimately obtained for this ion. This can be attributed to the extremely high variability of water quality in the Jukskei River. This is discussed in more detail in Section 5. Furthermore, because the results obtained did not contain at least two dates with measurements for both upstream and downstream vales, it was not possible to calculate a p-value for this dataset, and therefore the results are not statistically significant and cannot be used to calculate the average pollutant concentration increase across the Jukskei River.

4.3.8 Chloride

Figure 59 shows the results from tests measuring chloride concentration in samples from the Jukskei River, upstream and downstream of Alexandra.

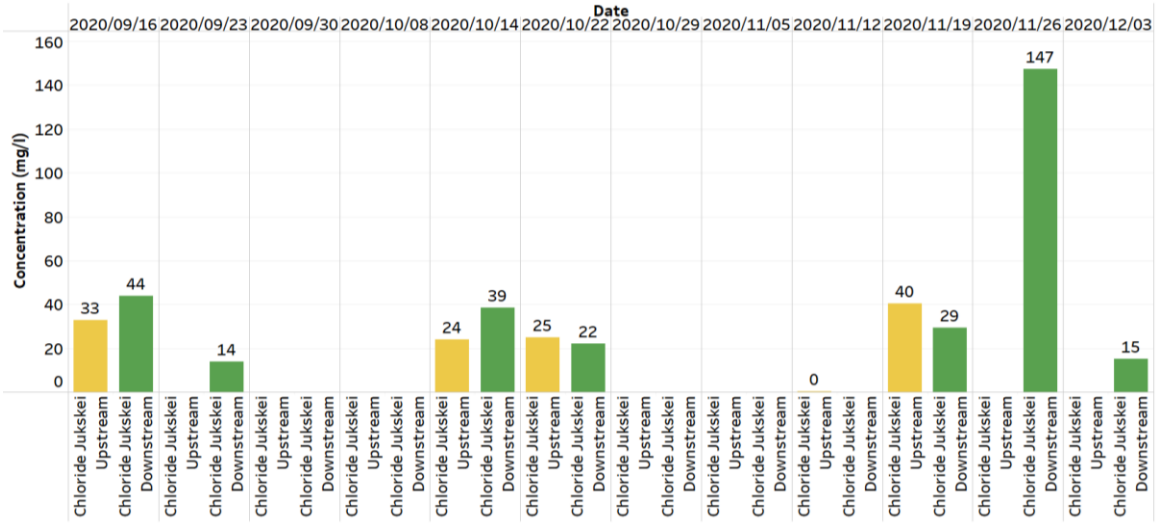

Figure 59: Results from chloride concentration (mg/l) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

The chloride concentration results for some weeks were beyond the detection limit of the ion chromatography machine used, and this is shown in Figure 59 by omitting the bar for that date.

Table 43 shows the statistical summary of the results that are graphed in Figure 59.

Table 43: Statistical summary of results obtained from chloride concentration (mg/l) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

	Chloride Jukskei Upstream	Chloride Jukskei Downstream
Mean (mg/l)	31	33
Min (mg/l)	24.1	22.1
Max (mg/l)	40.3	43.8
Range (mg/l)	16.2	21.7
Coefficient of Variation	24.8	28.9
p-value from paired t-test	0.66	

The difference in the means has a p-value of 0.66 (which is greater than 0.05) which means there is no statistically significant difference between upstream and downstream values. Therefore, these are results are not analysed any further.

4.3.9 Nitrate

Figure 60 shows the results from tests measuring nitrate concentration in samples from the Jukskei River, upstream and downstream of Alexandra.

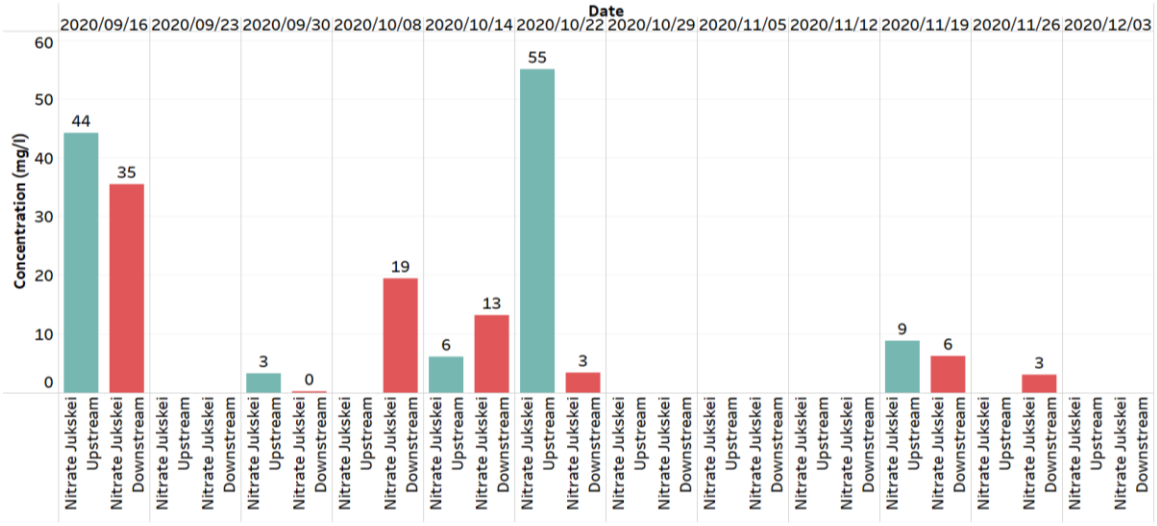

Figure 60: Results from nitrate concentration (mg/l) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

The nitrate concentration results for several weeks were beyond the detection limit of the ion chromatography machine used, and this is shown in by omitting the bar for that date.

Table 44 shows the statistical summary of the results that are graphed in Figure 60.

Table 44: Statistical summary of results obtained from nitrate concentration (mg/l) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

	Nitrate Jukskei Upstream	Nitrate Jukskei Downstream
Mean (mg/l)	23	12
Min (mg/l)	3.3	0.2
Max (mg/l)	55.0	35.5
Range (mg/l)	51.7	35.3
Coefficient of Variation	103.1	120.8
p-value from paired t-test	0.31	

The difference in the means has a p-value of 0.31 (which is greater than 0.05) which means there is no statistically significant difference between upstream and downstream values. Therefore, these are results are not analysed any further.

4.3.10 Nitrite

Figure 61 shows the results from tests measuring nitrate concentration in samples from the Jukskei River, upstream and downstream of Alexandra.

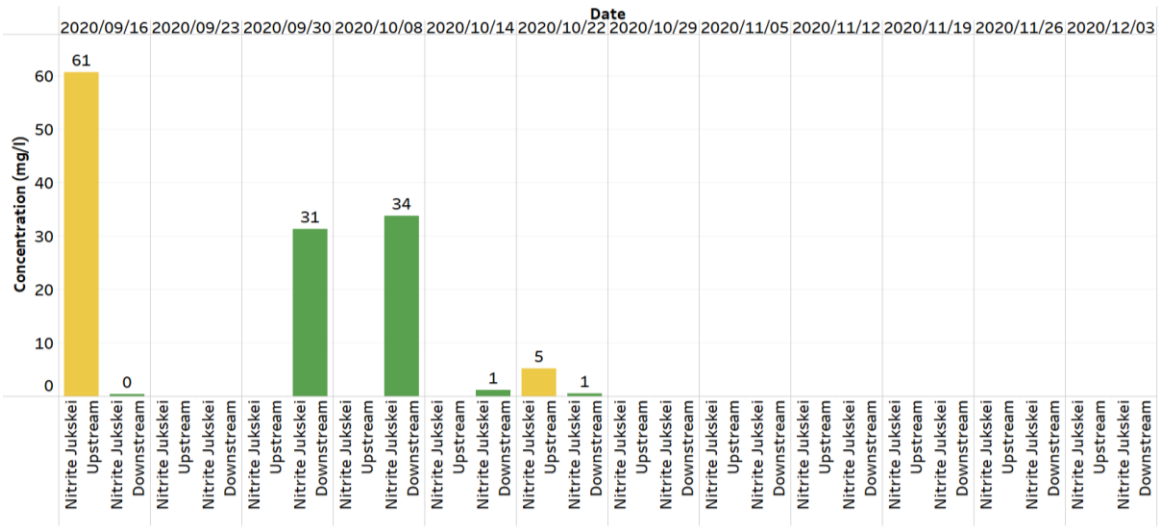

Figure 61: Results from nitrite concentration (mg/l) tests conducted on Results from nitrite concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 2

The nitrite concentration results for several weeks were beyond the detection limit of the ion chromatography machine used, and this is shown in Figure 61. by omitting the bar for that date. Although the researcher made every effort to obtain nitrite results by adjusting dilution concentrations for the sample placed in the ion chromatography machine and following the recommendations from the lab assistants, not many results were ultimately obtained for this ion. This can be attributed to the extremely high variability of water quality that was flowing through the wetlands. This is discussed in more detail in Section 5.

Table 45 shows the statistical summary of the results that are graphed in Figure 61.

Table 45: Statistical summary of results obtained from nitrite concentration (mg/l) tests conducted on Results from nitrite concentration (mg/l) tests conducted on influent and effluent streams of Constructed Wetland 2

	Nitrite Jukskei Upstream	Nitrite Jukskei Downstream
Mean (mg/l)	33	0
Min (mg/l)	5.2	0.4
Max (mg/l)	60.6	0.5
Range (mg/l)	55.4	0.2
Coefficient of Variation	118.9	28.0
p-value from paired t-test	0.45	

The difference in the means has a p-value of 0.45 (which is greater than 0.05) which means there is no statistically significant difference between upstream and downstream values. Therefore, these are results are not analysed any further.

4.3.11 Phosphate

Figure 62 shows the results from tests measuring phosphate concentration in samples from the Jukskei River, upstream and downstream of Alexandra.

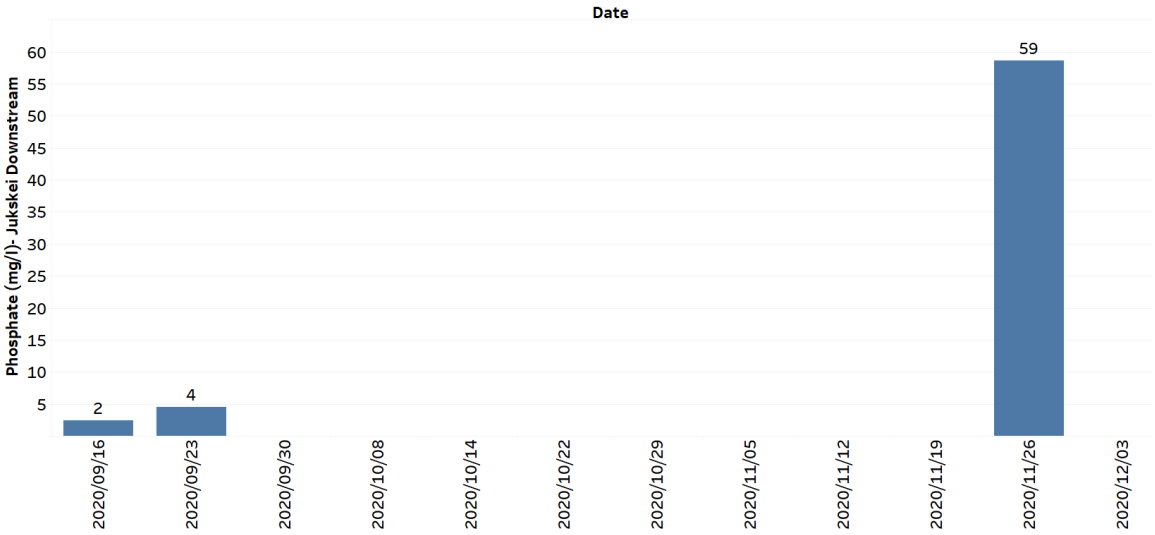

Figure 62: Results from phosphate concentration (mg/l) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

The phosphate concentration results for all upstream samples were beyond the detection limit of the ion chromatography machine used. Although the researcher made every effort to obtain phosphate results by adjusting dilution concentrations for the sample placed in the ion chromatography machine and following the recommendations from the lab assistants, not many results were ultimately obtained for this ion. This can be attributed to the extremely high variability of water quality that was flowing through the wetlands. This is discussed in more detail in Section 5. Furthermore, because the results obtained did not contain at least two dates with measurements for both inlet and outlet vales, it was not possible to calculate a p-value for this dataset, and therefore the results are not statistically significant and cannot be used to calculate the average pollutant concentration increase across the Jukskei River.

4.3.12 Sulfate

Figure 63 shows the results from tests measuring phosphate concentration in samples from the Jukskei River, upstream and downstream of Alexandra.

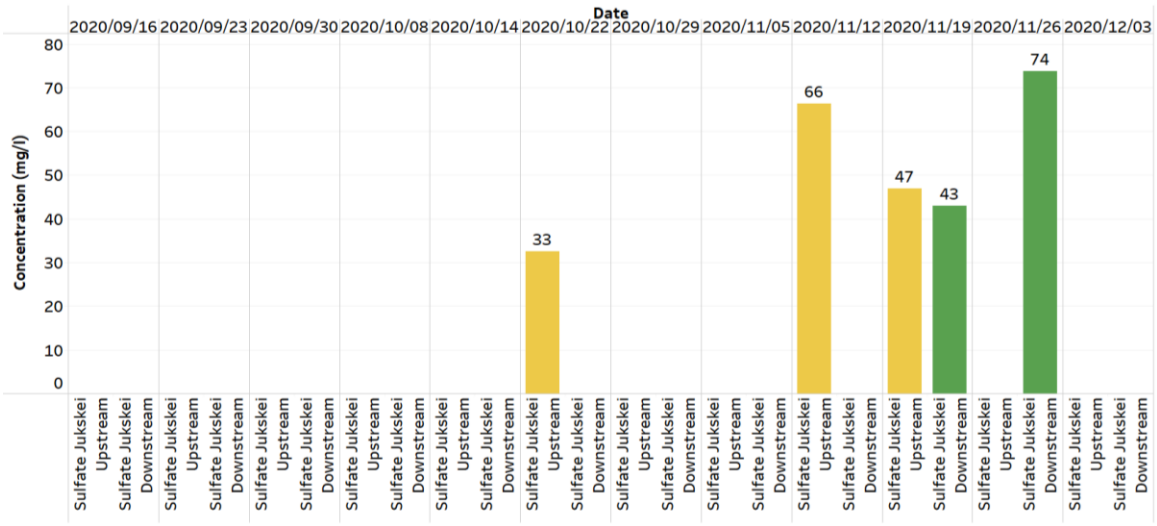

Figure 63: Results from sulfate concentration (mg/l) tests conducted on samples from the Jukskei River upstream and downstream of Alexandra

The sulfate concentration results for all upstream samples were beyond the detection limit of the ion chromatography machine used. Although the researcher made every effort to obtain sulfate results by adjusting dilution concentrations for the sample placed in the ion chromatography machine and following the recommendations from the lab assistants, not many results were ultimately obtained for this ion. This can be attributed to the extremely high variability of water quality that was flowing through the wetlands. This is discussed in more detail in Section 5. Furthermore, because the results obtained did not contain at least two dates with measurements for both inlet and outlet vales, it was not possible to calculate a p-value for this dataset, and therefore the results are not statistically significant and cannot be used to calculate the average pollutant concentration increase across the Jukskei River.

5 Discussion & Conclusion

5.1 Review of Hypotheses and Objectives

This study sought primarily to determine the effectiveness of constructed wetlands in wastewater treatment in an informal settlement. As discussed, constructed wetlands can potentially offer a low-cost, low-maintenance solution to wastewater treatment in areas that lack electricity, water engineers and basic delivery services from the municipality.

The hypotheses that were tested in this study were:

1. The constructed wetlands under study lower the concentration of coliform bacteria in domestic wastewater from Alexandra.
2. The constructed wetlands under study lower the concentration of chemical oxygen demand in domestic wastewater from Alexandra.
3. The constructed wetlands under study lower the concentration of nutrient ions in domestic wastewater from Alexandra.
4. The constructed wetlands under study improve in-situ parameters of domestic wastewater from Alexandra. For pH, the improvement means the water becomes more circumneutral and for electrical conductivity improvement a decrease in the value. For oxidation-reduction potential the constructed wetlands will decrease the value.
5. Constructed wetlands can act as buffers of water quality, lowering the range of concentration/value of the pollutant/parameter.
6. The water quality of the Jukskei River deteriorates between London Road, Alexandra (upstream of Alexandra) and Frankenwald, Sandton (downstream of Alexandra).

These hypotheses were tested through the following objectives for this study, that were approved by the University of Witwatersrand in the proposal that was submitted for this study.

Objectives of this study:

1. To conduct an ongoing characterisation of water from the Jukskei River, both upstream and downstream of Alexandra, to better analyse the effect of anthropogenic pollution from the township on the river.

2. To design, construct and commission a set of constructed wetlands in Alexandra using, to the extent that is feasible, locally-procurable materials.
3. To analyse the efficacy of the constructed wetland in treating domestic wastewater following successful construction.

This section includes separate subsections for each of the three systems under study, namely: constructed wetland 1, constructed wetland 2 and the Jukskei River. Each of these sub sections will explore the objectives that were achieved for the system under study, and discuss the results obtained and presented in Section 4.

5.2 General Considerations Pertinent to the Discussion of the Results obtained from the Constructed Wetlands

Before discussing the results obtained from the CWs, it is pertinent to discuss several factors that were observed in this study that affected the results and are important to bear in mind when interpreting the results. Many of these factors are based on this study being conducted in the field in an informal settlement in South Africa, rather than being conducted in a very controlled environment such as a laboratory.

5.2.1 Hydraulic Retention Time

One of the parameters that was not possible to measure in this study was the hydraulic retention times of the CWs. This parameter greatly determines the pollutant removal efficiency of the CW, particularly because one of the main pollutant removal mechanisms is through biological treatment of organic matter, and this biological treatment process requires a certain amount of time to be effective. For example, to achieve a high rate of coliform bacteria removal (more than 90%), it is generally reported in the literature that at least two days of hydraulic retention time is required in the CW (Khatiwada & Polprasert, 1999).

Although it was not possible to measure hydraulic retention time in the CWs in the field, some of the results obtained can be used to approximate a calculated hydraulic retention time, based on several assumptions, and this is discussed more further in this section.

5.2.2 Variability of Flow Rate

One of the key factors affecting the hydraulic retention times of the CWs, is the flow rate of wastewater entering the CWs. In this study, a very high variability of inlet flow rate was observed. This flow rate variability is due to the inherent peaks and troughs of the nature

of wastewater disposal in informal settlements. Often, as observed in this study, certain days of the week are and certain times of the day are reserved by residents for activities such as washing clothes in large buckets, cleaning homes, and washing dishes. This leads to large buckets of wastewater being disposed into the CWs all at once. These surges of wastewater can disrupt and displace microbial colonies in the CWs, and lead to a reduced pollutant removal efficiency until the bacteria are able to proliferate once more (Kadlec et al., 1995). Although it was not possible to quantify the variability of flow into the CWs in this field study, the pattern of wastewater disposal that was observed by this researcher, and other researchers working on this project, over numerous site visits, suggests that this is a factor which significantly affects the readings obtained.

5.2.3 Variability of Wastewater Quality

Relating to the variability in the flow rate of wastewater entering the CWs, is the high variability of the quality of that water. As discussed in the literature review, wastewater from informal settlements can range from being purely greywater, to purely blackwater as well as anywhere on the spectrum in-between these two qualities. Although the tests for *E. coli* in influent and effluent water did not produce results that were statistically significant to suggest a difference in the mean between inlet and outlet values, the detection of high concentrations in both of these streams is a confirmation of the presence of faecal contamination in the wastewater discarded into the CWs. This is in agreeance with the literature on wastewater arising in informal settlements than reports that often greywater is contaminated with blackwater (Oteng-Peprah et al., 2018).

The wastewater that residents discharged into the CWs came from many sources including dishwashing, clothes washing, cleaning the home, toilet water use, and often could be a combination of several of these sources. In some instances, it was even observed that residents would leave the tap running to fill up a bucket for washing, and then leave for several minutes, resulting in the bucket overflowing with water for that time, with all the clean municipal water from the tap running directly into the inlet of the CW. If there is little mixing of water occurring in the CW, this can result in a ‘plug’ of municipal water flowing through the wetland that can greatly affect results and could explain why for some readings, the inlet values were less polluted than the outlet values. However, even if there is a high level of mixing occurring in the wetland, this municipal water would dilute the surround body of water and affect results.

5.2.4 Evapotranspiration

As discussed, the location of the CWs that were built and monitored in this study is in an informal settlement in Alexandra, Johannesburg. This particular area has very few trees or tall buildings, and therefore receives harsh direct sunlight for most of the day. This, in turn, contributes to a very high rate of evapotranspiration in the CWs. Although it was not possible to quantify the rate of evapotranspiration in the field, it is safe to assume that evapotranspiration played a significant role in affecting the results, leading to a net concentration effect for some parameters such as total dissolved solids (measured in this study by the closely related parameter of electrical conductivity). This is supported by the literature, with some studies reporting evapotranspiration rates of up to 60% of influent water for constructed wetlands in developing countries (Kivaisi, 2001). Figure 64 shows the location where CW 1 was built, with the harsh direct sun, in this case necessitating that the researchers from Germany wear solar protective gear.



Figure 64: The location where Constructed Wetland 1 was built, showing the harsh direct sunlight that is typical of this area, in this case necessitating that the German researchers visiting wear adequate solar protective equipment

5.2.5 Short Circuiting of Flow

Another factor that no doubt affected the results in this study, was the short circuiting of wastewater flow in the CWs. As initially designed, the CWs would receive water in the inlet, this water would then be treated by the CW and release at the outlet. However, on many occasions, it was observed that some residents would discharge water from their buckets directly into the middle of the wetland. This could be due to the fact that the inlet was a bit raised from the ground, and therefore more difficult to dispose water into from a large bucket. It could also be due to some residents not wanting to walk to the inlet of the constructed wetland with a bucket full of water, and therefore just disposing it in the wetland at the closest point from their front doors. In any case, this factor can greatly affect the results, and can explain why some of the results obtained showed higher concentrations of pollutants at the outlet of the CWs. Figure 65 shows an informal house that was built on CW 1 with a resident reticulating their drainage pipe directly in the middle area of the wetland, bypassing the inlet area.



Figure 65:Constructed Wetland 1 showing an informal house that was built on one of its retaining walls with a drain leading into the middle area of the wetland and bypassing the inlet

5.2.6 Other Considerations

Several other factors were also observed in this field that that could affect the results obtained, although to a lesser degree than the aforementioned factors.

As the CWs were built in an area with very little service delivery, when a sewer would burst, it would take several weeks or months to be repaired. In this case, a sewer pipe from the neighbouring Malboro area would often burst and would discharge sewage along the road that would eventually flow adjacent to the retaining wall of CW 1. Although the researcher did not observe this raw sewage overflowing into the wetland on any field visits, the level of this sewage flowing adjacent to the wetland was often quite high, and it is likely that with a surge in flow, or with some rain, the sewage could have entered the CW on some occasions and affected results. This factor did not likely affect the results to a large extent as the overflow of sewage into the CW may have only occurred on one or two occasions. Shows CW 1 on the 29th of July 2020 with leaking sewage flowing adjacent to the retaining wall at a relatively high level.



Figure 66: View of Constructed Wetland 1 on the 29th of July 2020 showing leaking sewage flowing adjacent to the retaining wall at a relatively high level. Two young girls are using one of the retaining walls as a bridge to avoid the leaking sewage.

5.3 Discussion on Results from Constructed Wetland 1

Constructed wetland 1 was the first constructed wetland to be built in this study. Objective 2, namely to design, construct and commission a set of constructed wetlands in Alexandra using, to the extent that is feasible, locally-procurable materials; was achieved for this wetland. In addition, objective 1, namely to conduct an ongoing characterisation of water from the Jukskei River, both upstream and downstream of Alexandra, to better analyse the effect of anthropogenic pollution from the township on the river; was also achieved by carrying out a 12-week sampling campaign including weekly testing of inlet and outlet streams for the wetland.

Table 46 summarizes the parameters that were tested for constructed wetland 1 and whether the results from these tests were statistically significant or not.

Table 46: Summary of parameters that were tested for constructed wetland 1 and their statistical significance

Parameter	**Statistically Significant Results (Yes/No)**
Chemical Oxygen Demand (COD)	No
Total Coliforms	Yes
Escherichia coli	No
Electrical Conductivity	No
Oxidation-Reduction Potential (ORP)	Yes
pH	Yes
Fluoride	No
Chloride	No
Nitrate	No
Nitrite	No
Phosphate	Yes
Sulfate	No

As can be seen in Table 46, of the twelve parameters that were tested in inlet and outlet streams of CW 1, four of the parameters, namely total coliforms, ORP, pH and phosphate concentration yielded results that were statistically significant. As discussed in Section 3.3.6, the test of statistical significance that was used was the paired t-test. The paired t-test tests for the null-hypothesis i.e., that the true mean between the paired samples is zero. The four parameters with statistically significant results had p-values less than 0.05, indicating

that there is more than a 95% certainty that the null hypotheses can be ruled out, and that there is therefore a statistically significant difference between the means of the inlet and outlet values for these parameters in CW 1.

The following four subsections discuss the results of the four parameters that yielded statistically significant results for CW 1, namely total coliforms, ORP, pH and phosphate respectively.

5.3.1 Total Coliforms

Total coliforms measured in the inlet and outlet streams of CW 1 yielded an extremely low p-value of 0.000047 for the paired t-test, which indicates that there is a 99.995% confidence that the difference of mean inlet and outlet values is not zero.

The mean inlet value for total coliforms in CW 1 was 1.38E+11 CFU/100ml, and the mean outlet value was 2.07E+10 CFU/100ml. Concentration of total coliforms in the wastewater entering CW 1 was 5 orders of magnitude greater that the concentration of total coliforms reported in greywater in peri urban areas of developing countries that is reported in the literature (Oteng-Peprah et al., 2018).

The difference between inlet and outlet concentrations for total coliforms in CW 1 equates to a reduction of 85%. This result is in accordance with other results reported by Gerba et al. (1999), Khatiwada & Polprasert (1999) and Thurston et al. (2001) who reported coliform reduction rates of up to 99% in both free water surface constructed wetlands and vegetated submerged bed CWs, the latter being the type of CWs constructed in this study. Coliform bacteria removal has also been shown to be assisted by ultraviolet radiation in free water surface CWs, and this could be one reason that the reduction achieved in this study is not as high as 99% (Mara, 1976). Another reason that could contribute to the coliform bacteria reduction level achieved in this study being lower than that reported in some of the literature, is that the reduction of up to 99% was reported for CW with a minimum retention time of three days. As discussed, constructing and monitoring a CW in the field in an informal settlement is vastly different from conducting such studies in the laboratory, and therefore the hydraulic retention time of the system could not be measured in this study. However, these results indicate that the hydraulic retention time of CW 1 is likely less than 3 days.

Based on the comparison of the coliform reduction rate of CW 1 to the literature, a very rough calculation can be made to give a ‘ballpark’ figure for the flowrate of wastewater

through the wetland. Referring to the technical drawing of CW 1 presented in Section 3.2.4, the approximate internal volume of the wetland is calculated by Equation 20.

Equation 20: Equation for the calculation of the volume of CW 1

$$(300 + 240 + 2770) \times (1020) \times (500) \times 10^{-9} = 1.688\, m^3$$

This internal volume of CW 1 when it is empty can then be multiplied by the void ratio for gravel (0.4) to yield the approximate volume of water in the wetland, as shown in Equation 21.

Equation 21: Equation for the calculation of the volume of water in CW 1

$$1688 \times 0.4 = 675\, l$$

This volume of internal water can be divided by the hydraulic retention time of 3 days and divided by the percent difference in the reduction of coliform bacteria that was achieved in this study versus that which was achieved in the literature, to obtain a very approximate flowrate of wastewater flowing through CW 1, as shown in Equation 22.

Equation 22: Rough calculation of approximate flowrate of wastewater through CW 1 based on comparing the reduction of total coliforms of CW 1 to the literature

$$675 \div 3 \div (\frac{85}{99}) \approx 200\, l/day$$

This indicates that a very rough approximation of the flow rate of wastewater in CW 1, based on a comparison of the coliform reduction achieved (85%), compared to that reported in the literature (99%), is 200 l/day. The researcher would like to emphasize that this is a very rough approximation, based on several assumptions including that a linear scaling can be used to compare the hydraulic retention time of one CW to another, and this result would need to be verified by field tests and further monitoring.

Furthermore, the results obtained for total coliforms from CW 1, indicate that there is not only a net reduction in coliform bacteria across the wetland, but there is also a reduction in the range of concentration of total coliforms across the wetland. The range in total coliform concentration for inlet values in CW 1 was 4.8392E+11 CFU/100ml, while the range for the outlet was 7.70E+10 CFU/100ml, a reduction of one order of magnitude. This supports the premise that constructed wetlands can act as buffers that reduce the variability of water quality flowing through a system (Mander et al., 2017).

5.3.2 Oxidation-Reduction Potential

Oxidation-Reduction Potential (ORP) measured in the inlet and outlet streams of CW 1 yielded an extremely low p-value of 0.000047 for the paired t-test, which indicates that there is a 99.998% confidence that the difference of mean inlet and outlet values is not zero.

The mean value for ORP in the inlet of CW 1 was -204mV while the mean outlet value was -302 mV. This indicates that there was a net reduction in ORP of approximately 48% across CW 1. Oxidation-reduction potential values in influent and effluent streams in this range indicate that of the biochemical reactions taking place in CW 1, the most prominent would include hydrogen sulfide formation, biological phosphorus release, acid formation and methane production (YSI, 2021). Although these reactions are important in treating wastewater, a reduction in ORP indicates that there is less dissolved oxygen in the effluent water than in the influent water, and this can have an adverse effect on the receiving Jukskei River as most aquatic system should contain approximately 6-8mg/l of dissolved oxygen to be able to sustain flora and fauna (Wallace & Knight, 2006).

It was also observed that all samples of wastewater collected from the inlet to CW 1 were negative in ORP. This is typical for highly concentrated domestic wastewater as the level of dissolved oxygen and other oxidants is generally very low (Arden & Ma, 2018). Observing a negative ORP that is lower than influent ORP aligns with results from other studies that show that CWs, and in particular vegetated submerged bed designs of CWs, contain anaerobic conditions that are reductive (Oteng-Peprah et al., 2018; Wallace & Knight, 2006)

The range in ORP in effluent water from CW 1 was also approximately 40% less than the range of ORP in influent water to CW 1 (248.8 mV in influent vs 148.5 mV in effluent). This supports the premise that constructed wetlands can act as buffers that reduce the variability of water quality flowing through a system (Mander et al., 2017).

Oxidation-reductio potential, is seldom measured in studies researching constructed wetlands for domestic wastewater treatment, perhaps because a special probe is required for its measurement, and hence no other studies were found with which to compare these results.

5.3.3 pH

Values for pH measured in the inlet and outlet streams of CW 1 yielded a relatively low p-value of 0.0054 for the paired t-test, which indicates that there is a 99.46% confidence that the difference of mean inlet and outlet values is not zero.

The mean pH values for inlet and outlet streams of CW 1 were 7.2 and 7.9 respectively. This represents a net increase in the level of pH across the wetland and indicates that there is a reduction in the concentration of hydrogen ions in solution as the wastewater is treated in the wetland.

The range of pH values recorded for the wastewater inlet to CW 1 (pH of 5.9 – 8.5), correlates very closely to values for pH in greywater from peri urban areas in developing countries reported in the literature (pH of 5-8) (Oteng-Peprah et al., 2018). Furthermore, the pH of influent entering CW 1 varied between below neutral and above neutral pH, whereas the pH of effluent water from CW 1 was all recorded above neutral. It is generally acknowledged that a pH that is closer to neutral and preferably slight above neutral, is more hospitable for organic life (Wallace & Knight, 2006). In addition, the range of pH of effluent water from CW 1 (range of 0.84) was significantly lesser than that for the influent water (range of 2.58). This supports the premise that constructed wetlands can act as buffers that reduce the variability of water quality flowing through a system (Mander et al., 2017).

These results support hypotheses 4 of this study, namely that the constructed wetlands under study improve in-situ parameters including pH, electrical conductivity, and oxidation-reduction potential in domestic wastewater from Alexandra.

5.3.4 Phosphate

Phosphate concentration measured in the inlet and outlet streams of CW 1 yielded a p-value of 0.03 for the paired t-test, which indicates that there is a 97% confidence that the difference of mean inlet and outlet values is not zero.

The mean inlet concentration for phosphate in CW 1 was 69 mg/l, whereas the mean outlet concentration was 17mg/l. This represents a pollutant removal efficiency of approximately 75% for phosphate. This pollutant removal efficiency is slightly higher than what is reported in the literature of 71% for phosphate (Oteng-Peprah et al., 2018). The main mechanisms that remove phosphate in a constructed wetland are plant nutrient uptake (but only in the first few seasons of the plants growth), sedimentation and adsorption (Wallace & Knight, 2006). Because the samples collected were from the constructed wetland within

the first growing season of the reeds that were plant, it is likely that plant nutrient uptake played a significant role in reducing the concentration of phosphate in the wastewater.

The range of phosphate concentration for effluent water from CW 1 (27.5 mg/l) was approximately 80% lower than the range of phosphate concentration in influent water (136.9 mg/l). This supports the premise that constructed wetlands can act as buffers that reduce the variability of water quality flowing through a system (Mander et al., 2017).

5.4 Discussion on Results from Constructed Wetland 2

Constructed wetland 2 was the second constructed wetland to be built in this study. Objective 2, namely to design, construct and commission a set of constructed wetlands in Alexandra using, to the extent that is feasible, locally-procurable materials; was achieved for this wetland. In addition, objective 1, namely to conduct an ongoing characterisation of water from the Jukskei River, both upstream and downstream of Alexandra, to better analyse the effect of anthropogenic pollution from the township on the river; was also achieved by carrying out a 12-week sampling campaign including weekly testing of inlet and outlet streams for the wetland.

Table 47 summarizes the parameters that were tested for constructed wetland 2 and whether the results from these tests were statistically significant or not.

Table 47: Summary of the parameters that were tested for constructed wetland 2 and their statistical significance

Parameter	**Statistically Significant Results (Yes/No)**
Chemical Oxygen Demand (COD)	No
Total Coliforms	Yes
Escherichia coli	No
Electrical Conductivity	Yes
Oxidation-Reduction Potential (ORP)	No
pH	No
Fluoride	No
Chloride	No
Nitrate	No
Nitrite	No
Phosphate	Yes
Sulfate	No

As can be seen in Table 47, of the twelve parameters that were tested in inlet and outlet streams of CW 2, three of the parameters, namely total coliforms, electrical conductivity and phosphate concentration yielded results that were statistically significant. As discussed in Section 3.3.6, the test of statistical significance that was used was the paired t-test. The paired t-test tests for the null-hypothesis i.e., that the true mean between the paired samples is zero. The three parameters with statistically significant results had p-values less than 0.05, indicating that there is more than a 95% certainty that the null hypotheses can be ruled out, and that there is therefore a statistically significant difference between the means of the inlet and outlet values for these parameters in CW 2.

5.4.1 Total Coliforms

Total coliforms measured in the inlet and outlet streams of CW 2 yielded a low p-value of 0.0012 for the paired t-test, which indicates that there is a 99.88% confidence that the difference of mean inlet and outlet values is not zero.

The mean inlet value for total coliforms in CW 2 was 8.59E+10 CFU/100ml, and the mean outlet value was 6.96E+9 CFU/100ml. Concentration of total coliforms in the wastewater entering CW 2 was four orders of magnitude greater that the concentration of total coliforms reported in greywater in peri urban areas of developing countries that is reported in the literature (Oteng-Peprah et al., 2018).

The difference between inlet and outlet concentrations for total coliforms in CW 2 equates to a reduction of 92%. This result is in accordance with other results reported by Gerba et al. (1999), Khatiwada & Polprasert (1999) and Thurston et al. (2001) who reported coliform reduction rates of up to 99% in both free water surface constructed wetlands and vegetated submerged bed CWs, the latter being the type of CWs constructed in this study. Coliform bacteria removal has also been shown to be assisted by ultraviolet radiation in free water surface CWs, and this could be one reason that the reduction achieved in this study is not as high as 99% (Mara, 1976). Another reason that could contribute to the coliform bacteria reduction level achieved in this study being lower than that reported in some of the literature, is that the reduction of up to 99% was reported for CW with a minimum retention time of three days. As discussed, constructing and monitoring a CW in the field in an informal settlement is vastly different from conducting such studies in the laboratory, and therefore the hydraulic retention time of the system could not be measured in this study.

However, these results indicate that the hydraulic retention time of CW 2 is likely to be slightly less than 3 days.

Based on the comparison of the coliform reduction rate of CW 2 to the literature, a very rough calculation can be made to give a 'ballpark' figure for the flowrate of wastewater through the wetland. Referring to the technical drawing of CW 2 presented in Section 3.2.4, the approximate internal volume of the wetland is calculated by Equation 23.

Equation 23: Equation for the calculation of the volume of CW 2

$$(300 + 240 + 1560) \times (1020) \times (500) \times 10^{-9} = 1.071\ m^3$$

This internal volume of CW 2 when it is empty can then be multiplied by the void ratio for gravel (0.4) to yield the approximate volume of water in the wetland, as shown in Equation 24.

Equation 24: Equation for the calculation of the volume of water in CW 2

$$1071 \times 0.4 = 428\ l$$

This volume of internal water can be divided by the hydraulic retention time of 3 days and divided by the percent difference in the reduction of coliform bacteria that was achieved in this study versus that which was achieved in the literature, to obtain a very approximate flowrate of wastewater flowing through CW 2, as shown in Equation 25.

Equation 25: Rough calculation of approximate flowrate of wastewater through CW 1 based on comparing the reduction of total coliforms of CW 2 to the literature

$$428 \div 3 \div (\frac{92}{99}) \approx 133\ l/day$$

This indicates that a very rough approximation of the flow rate of wastewater in CW 2, based on a comparison of the coliform reduction achieved (92%), compared to that reported in the literature (99%), is 133 l/day. The researcher would like to emphasize that this is a very rough approximation, based on several assumptions including that a linear scaling can be used to compare the hydraulic retention time of one CW to another, and this result would need to be verified by field tests and further monitoring.

Furthermore, the results obtained for total coliforms from CW 2, indicate that there is not only a net reduction in coliform bacteria across the wetland, but there is also a reduction in the range of concentration of total coliforms across the wetland. The range in total coliform concentration for inlet values in CW 2 was 4.8392E+11 CFU/100ml, while the range for

the outlet was 3.45E+10 CFU/100ml, a reduction of one order of magnitude. This supports the premise that constructed wetlands can act as buffers that reduce the variability of water quality flowing through a system (Mander et al., 2017).

5.4.2 Electrical Conductivity

Electrical conductivity (EC) measured in the inlet and outlet streams of CW 2 yielded a p-value of 0.026 for the paired t-test, which indicates that there is a 97.4% confidence that the difference of mean inlet and outlet values is not zero.

The mean inlet value of EC in CW 2 was 2480 µS/cm whilst in the outlet it was 3269 µS/cm. This indicates a net increase of approximately 32% in EC across CW 2. This is likely due to the very high rate of evapotranspiration in Alexandra. Evapotranspiration leads to a net concentrating effect of the wastewater. As electrical conductivity is primarily a measure of the concentration of ions in the water, this concentration would increase as the volume of water decreases because of evaporation.

The mean inlet EC of 2480 µS/cm, is approximately 21% higher than the EC reported in greywater from peri urban areas in developing countries (2044 µS/cm) (Oteng-Peprah et al., 2018). This is in conformity with results from the other parameters tested in this study which were also more concentrated than the values reported in the literature.

The range of values for electrical conductivity in effluent water from CW 2 (2283 µS/cm) was approximately 48% lower than the range of phosphate concentration in influent water (2283 µS/cm). This supports the premise that constructed wetlands can act as buffers that reduce the variability of water quality flowing through a system (Mander et al., 2017).

5.4.3 Phosphate

Phosphate concentration measured in the inlet and outlet streams of CW 2 yielded a p-value of 0.01 for the paired t-test, which indicates that there is a 99% confidence that the difference of mean inlet and outlet values is not zero. However, this p-value was based only on readings that were measured for two dates, as the readings from the third date for which results were obtained had to be discarded as they contained a distant outlier. The researcher does not feel that two dates of data is a sufficient sample size to draw conlusions from, out of a possible twelve dates of sampling, however a brief discussion on this data is included for the sake of completeness and objectivity in the statistical analysis.

The mean inlet concentration for phosphate in CW 2 was 5 mg/l, whereas the mean outlet concentration was 11mg/l. This represents a pollutant removal efficiency of approximately -120% for i.e. that there was a net increase in phosphate concentration across the wetland. The main mechanisms that remove phosphate in a constructed wetland are plant nutrient uptake (but only in the first few seasons of the plants growth), sedimentation and adsorption (Wallace & Knight, 2006). A possible reason for this apparent increase in the concentration of phosphate across CW 2 could be due to residents discharging their wastewater into the middle of the CW and therefore short circuiting the CW hydraulically. Another possible explanation is that, because this wetland generally had less flow passing through it, certain 'dead spots' or 'pockets of stagnancy' were created where the water stagnated and did not mix with the rest of the water in the CW. If a sample of this stagnant water was sampled at the inlet, this would affect results as it would essentially have been hydraulically disconnected from the effluent water of CW 2.

The range of phosphate concentration for effluent water from CW 2 (7.9 mg/l) was slightly lower than the range of phosphate concentration in influent water (8.1 mg/l). This supports the premise that constructed wetlands can act as buffers that reduce the variability of water quality flowing through a system (Mander et al., 2017).

5.5 Discussion on Results from the Jukskei River

One of the objectives of this study, was to conduct an ongoing characterisation of water from the Jukskei River, both upstream and downstream of Alexandra, to better analyse the effect of anthropogenic pollution from the township on the river. This objective was met through carrying out a sampling campaign of river water for twelve weeks in the study. The results from carrying out this sampling campaign, were then used to test one of the hypotheses of this study, namely that the water quality of the Jukskei River deteriorates between London Road, Alexandra (upstream of Alexandra) and Kelvin, Sandton (downstream of Alexandra).

Table 48 summarizes the parameters that were tested from samples collected from the Jukskei River, and whether the results from these tests were statistically significant or not.

Table 48: Summary of the parameters that were tested for the Jukskei River and their statistical significance

Parameter	Statistically Significant Results (Yes/No)
Chemical Oxygen Demand (COD)	Yes
Total Coliforms	Yes

Escherichia coli	Yes
Electrical Conductivity	Yes
Oxidation-Reduction Potential (ORP)	Yes
pH	Yes
Fluoride	No
Chloride	No
Nitrate	No
Nitrite	No
Phosphate	No
Sulfate	No

As can be seen in Table 48, of the twelve parameters that were tested in upstream and downstream samples of the Jukskei River, six of the parameters, namely chemical oxygen demand, total coliforms, *E. coli*, electrical conductivity, oxidation-redox potential and pH yielded results that were statistically significant. As discussed in Section 3.3.6, the test of statistical significance that was used was the paired t-test. The paired t-test tests for the null-hypothesis i.e., that the true mean between the paired samples is zero. The six parameters with statistically significant results had p-values less than 0.05, indicating that there is more than a 95% certainty that the null hypotheses can be ruled out, and that there is therefore a statistically significant difference between the means of the upstream and downstream values for these parameters in the Jukskei River.

5.5.1 Chemical Oxygen Demand

Chemical oxygen demand measured in the Jukskei River upstream and downstream of Alexandra yielded a low p-value of 1.33E-8 for the paired t-test, which indicates that there is more than a 99.99% confidence that the difference of mean upstream and downstream values is not zero.

The mean upstream concentration for chemical oxygen demand in the Jukskei River was 28 mg/l, with the mean downstream concentration measured at 83 mg/l. This indicates a significant increase in COD concentration across the river of approximately 196%. This increase in COD can largely be attributed to anthropogenic pollution from Alexandra. On several occasions, this researcher followed different stream of wastewater that were originating in Alexandra to their outflow in the river to confirm that the domestic

wastewater that is generated in Silvertown, Alexandra is hydraulically connected to the Jukskei River. Figure 67 shows an example of one of the streams of wastewater flowing through Silvertown, Alexandra that was confirmed to be hydraulically connected to the Jukskei River.



Figure 67: An example of one of the wastewater streams in Silverton, Alexandra that was confirmed to be hydraulically connected to the Jukskei River

The mean downstream concentration of COD exceeds the legislated discharge standard of water to an open water source of 75 mg/l COD (Department of Water and Environmental Affairs of South Africa, 2013). This increased concentration of COD can deplete river water of dissolved oxygen and make the river water much less hospitable to local fauna and flora.

Downstream concentrations of COD in the Jukskei River also showed a higher range of concentrations (42mg/l downstream vs 11 mg/l upstream) and a higher coefficient of variation (19.1 downstream vs 15.3 upstream). This indicates that the anthropogenic pollution entering the Jukskei River is of variable quality.

Overall, results from the COD tests conducted on samples collected from the Jukskei support the hypothesis under study, namely that the water quality of the Jukskei River

deteriorates between London Road, Alexandra (upstream of Alexandra) and Kelvin, Sandton (downstream of Alexandra).

5.5.2 Total Coliforms

Total coliforms measured in the Jukskei River upstream and downstream of Alexandra yielded a low p-value of 1.03E-4 for the paired t-test, which indicates that there is more than a 99.99% confidence that the difference of mean upstream and downstream values is not zero.

The mean upstream concentration for total coliforms in the Jukskei River was 2.95E+8 CFU/100ml, with the mean downstream concentration measured at 3.03E+10 CFU/100ml. This indicates a significant increase in total coliform concentration across the river of approximately 10144%. As discussed, it is very clear that the source of this pollution in the Jukskei River is anthropogenic.

Downstream concentrations of total coliforms in the Jukskei River also showed a higher range of concentrations (1.05E+11 CFU/100ml downstream vs 6.63E+8 CFU/100ml upstream) and a higher coefficient of variation (80 downstream vs 67 upstream). This indicates that the anthropogenic pollution entering the Jukskei River is of variable quality.

Overall, results from the total coliform tests conducted on samples collected from the Jukskei support the hypothesis under study, namely that the water quality of the Jukskei River deteriorates between London Road, Alexandra (upstream of Alexandra) and Kelvin, Sandton (downstream of Alexandra).

5.5.3 Escherichia coli

Escherichia coli measured in the Jukskei River upstream and downstream of Alexandra yielded a low p-value of 2.21E-6 for the paired t-test, which indicates that there is more than a 99.99% confidence that the difference of mean upstream and downstream values is not zero.

The mean upstream concentration for *E. coli* in the Jukskei River was 1.90E+7 CFU/100ml, with the mean downstream concentration measured at 2.06E+9 CFU/100ml. This indicates a significant increase in *E. coli* concentration across the river of approximately 10727%. As discussed, it is very clear that the source of this pollution in the Jukskei River is anthropogenic.

It is also evident that the concentration of *E. coli* in the Jukskei is already four orders of magnitude larger that the legislated discharge standard of wastewater to an open body before it is polluted from wastewaters in Alexandra. This indicates that there is faecal contamination in the Jukskei River upstream of Alexandra. However, after the river water passes through Alexandra and is polluted further, the concentration of *E. coli* increases to 6 orders of magnitude greater than the legislated discharge standard.

Downstream concentrations of *E. coli* in the Jukskei River also showed a higher range of concentrations (4.3E+9 CFU/100ml downstream vs 5.8E+7 CFU/100ml upstream). This indicates that the anthropogenic pollution entering the Jukskei River is of variable quality.

Overall, results from the *E. coli* tests conducted on samples collected from the Jukskei support the hypothesis under study, namely that the water quality of the Jukskei River deteriorates between London Road, Alexandra (upstream of Alexandra) and Kelvin, Sandton (downstream of Alexandra).

5.5.4 Electrical Conductivity

Electrical conductivity measured in the Jukskei River upstream and downstream of Alexandra yielded a low p-value of 6.9E-4 for the paired t-test, which indicates that there is more than a 99.93% confidence that the difference of mean upstream and downstream values is not zero.

The mean upstream value for electrical conductivity in the Jukskei River was 420 µS/cm, with the mean downstream value measured at 538 µS/cm. This indicates an increase in electrical conductivity across the river of approximately 28%. As discussed, it is very clear that the source of this pollution in the Jukskei River is anthropogenic.

Electrical conductivity is primarily a measure of the concentration of ions in water, as a higher concentration of ions generally leads to a higher electrical conductivity of the water. Therefore, it is evident that the pollution entering the Jukskei River contains a relatively high concentration of ions and this is likely due to the use of cleaning products and the fact that greywater from informal settlements is often more concentrated due to bucket water being reused for various cleaning activities (Oteng-Peprah et al., 2018).

In contrast to the other parameters measured in samples from the Jukskei River, the range of electrical conductivity measured in upstream samples (267 µS/cm) was greater than downstream samples (201 µS/cm). The coefficient of variation of electrical conductivity in

upstream samples (22.5) was also greater than downstream samples (13.5). There are several possible reasons for this observation. This could be due to an upstream industrial polluter of the Jukskei River, that is discharging effluent of a variable electrical conductivity into the river, which increases that range of upstream electrical conductivity, but does not increase the range of downstream electrical by the same amount as the downstream water is diluted by wastewater from Alexandra. Another potential explanation for this observation is that the wastewater being discharged into the Jukskei River from Alexandra is of relatively consistent electrical conductivity. Further testing of upstream and downstream water, as well as surveys of nearby pollutants into the Jukskei River would need to be conducted to arrive at more definitive explanation for this observation.

Overall, results from the electrical conductivity tests conducted on samples collected from the Jukskei support the hypothesis under study, namely that the water quality of the Jukskei River deteriorates between London Road, Alexandra (upstream of Alexandra) and Kelvin, Sandton (downstream of Alexandra). All samples measure from both upstream and downstream of the Jukskei River were below the legislated discharge standard of wastewater to an open water body of 1500 µS/cm (Department of Water and Environmental Affairs of South Africa, 2013).

5.5.5 Oxidation-Redox Potential

Oxidation-redox potential (ORP) measured in the Jukskei River upstream and downstream of Alexandra yielded a p-value of 0.04 for the paired t-test, which indicates that there is a 96% confidence that the difference of mean upstream and downstream values is not zero.

The mean upstream value for oxidation-redox potential in the Jukskei River was 61 mV, with the mean downstream value measured at -45 mV. This indicates a decrease in oxidation-redox potential across the river of approximately 170%. As discussed, it is very clear that the source of this pollution in the Jukskei River is anthropogenic.

These results also show a clear switch that is occurring across the river from the river water being in an oxidative state upstream, with a positive ORP, to a reductive state downstream, with a negative ORP. This is indicative of a depletion of dissolved oxygen and other oxidants in the water which greatly interferes with the natural life cycles of aquatic flora and fauna (Wallace & Knight, 2006).

A negative ORP in downstream water can be explained by the high chemical oxygen demand in the wastewater that is being discharged into the river, which depletes dissolved

oxygen as well as other oxidants such as sodium hypochlorite that would be present in wastewater from the use of bleach in cleaning.

Similar to the measurements of electrical conductivity, the range of ORP in upstream samples (242 mV) was higher than the range of ORP in downstream samples (151). For this parameter, it was not possible to calculate the coefficient of variation, because the results contained both positive and negative numbers. Further testing of upstream and downstream water, as well as surveys of nearby pollutants into the Jukskei River would need to be conducted to arrive at more definitive explanation for this observation.

Overall, results from the oxidation-reduction potential tests conducted on samples collected from the Jukskei support the hypothesis under study, namely that the water quality of the Jukskei River deteriorates between London Road, Alexandra (upstream of Alexandra) and Kelvin, Sandton (downstream of Alexandra). Currently, there is no discharge standard for ORP in wastewater that is discharged to an open water body (Department of Water and Environmental Affairs of South Africa, 2013).

5.5.6 pH

Values for pH measured in the Jukskei River upstream and downstream of Alexandra yielded a p-value of 0.004 for the paired t-test, which indicates that there is a 99.6% confidence that the difference of mean upstream and downstream values is not zero.

The mean upstream value for pH in the Jukskei River was 7.89, with the mean downstream value measured at 7.65. Although this may seem a small decrease in pH, it is important to remember that pH measures the concentration of ions on the log scale. Converting these values of pH to the concentrations of hydrogen ions, results in a calculated increase of hydrogen ions of 74% across the river.

The range of pH values measured upstream (0.16) was less than the range of pH values measured downstream (0.24). Likewise, the coefficient of variation of pH in upstream samples (0.7) was less than half of that calculated from downstream samples (1.5). This indicates that the wastewater that is entering the Jukskei River from Alexandra is of a variable pH and contributes to a greater instability of pH in the river.

All results measured were within the legislated discharge standard for wastewater to an open water body of between 5.5 – 9.5 pH (Department of Water and Environmental Affairs of South Africa, 2013). Therefore, results from the pH test do not support the hypothesis

under study, namely that the water quality of the Jukskei River deteriorates between London Road, Alexandra (upstream of Alexandra) and Kelvin, Sandton (downstream of Alexandra).

5.6 Conclusion and Recommendations

In summary, all the three objectives of this study listed in Section 1.3 were met. The results from testing the hypotheses under study are discussed below.

Results from the study, discussed in Sections 5.3.1 and 5.4.1, were unable to disprove hypothesis 1, namely that the constructed wetlands under study reduce the concentration of Coliform bacteria in domestic wastewater from Alexandra.

Hypothesis 2: the constructed wetlands under study lower the concentration of chemical oxygen demand in domestic wastewater from Alexandra; was disproven by the results of the study, as there was no statistical significance between inlet and outlet values for COD in CW 1 and CW 2. These results are likely affected by the factors discussed in Section 5.2, but for the purposes of this study, hypothesis 2 was disproven by the results obtained.

Hypothesis 3: the constructed wetlands under study lower the concentrations of nutrient ions in domestic wastewater from Alexandra; was not disproven by some the results from this study. As discussed in Sections 5.3.4, results showed that there was a removal of phosphate ion concentration across CW 1. Results discussed in Section 5.4.3, however, showed that there was an increase in the concentration of phosphate ions across CW 2. Overall, this hypothesis cannot be supported, and further testing is required to come to a more conclusive result. Therefore, the hypothesis was not disproven for CW 1, but was disproven for CW 2. Overall, the results are inconclusive.

Hypothesis 4: the constructed wetlands under study improve in-situ parameters of domestic wastewater from Alexandra. For pH, the improvement means the water becomes more circumneutral and for electrical conductivity improvement a decrease in the value. For oxidation-reduction potential the constructed wetlands will decrease the value. The results were inconclusive for pH since we were able to disprove the hypothesis for CW 1 but not CW 2. For electrical conductivity, the hypothesis was disproven for CW 2 but not for CW 1. For ORP, the hypothesis was not disproven as ORP decreased in CW 1. Under these circumstances, we can see that the constructed wetlands function as separate systems with different treatment conditions. The inconclusive nature of these results is likely affected by the factor discussed in Section 5.2.

Hypothesis 5: constructed wetlands can act as buffers of water quality, reducing the range of concentration/value of the pollutant/parameter; was largely supported by the evidence. Most parameters tested for both constructed wetlands showed a smaller range and coefficient of variation for outlet samples compared to inlet samples. This hypothesis was not disproved by the results.

Hypotheses 6: the water quality of the Jukskei River deteriorates between London Road, Alexandra (upstream of Alexandra) and Frankenwald, Sandton (downstream of Alexandra); was supported by the results from this study, discussed in Section 5.5. Chemical oxygen demand, total coliforms, *E. coli*, electrical conductivity, oxidation-reduction potential and pH were all shown to deteriorate between upstream and downstream samples tested from the Jukskei River. Thus, this hypothesis was not disproven by the results.

The data found in this study indicate that informal settlements have a substantial and significant negative impact on the natural environment where they are located. Whilst the treatment systems in this study had variable effectiveness towards ameliorating this impact, there is still a desperate for sanitation interventions for the residents of these communities, not only for these residents but also for the environment since the rivers transport the impact to other communities.

Based on the work undertaken in this research study, several recommendations are made for future research and endeavours that are undertaken in this field.

Regarding the measuring and calculation of pollutant removal efficiencies, taking evapotranspiration into account would yield more accurate results as there is likely a pollutant concentration effect that is caused by evapotranspiration in the wetland. One approach to calculate evapotranspiration rates within a location like Silvertown could include measuring total dissolved solids values for both influent and effluent streams in the wetland and using the difference between the two concentrations to calculate the water lost to evapotranspiration. This would essentially be a first principles, mass balance approach.

Another recommendation pertaining to the accuracy and statistical representability of samples collected from the constructed wetlands, is to account for system retention time in

these measurements. Several sample sets that were collected from the constructed wetlands, presented in Section 4, seem to indicate that there is a negative pollutant removal efficiency in the wetlands. This is likely due to several factors, including surges of different quality wastewater that is flowing through the constructed wetlands in a plug-flow regime. Without taking these plugs of different quality water into account, results could understate the effectiveness of constructed wetland in removing pollutants from wastewater.

Furthermore, increasing the frequency of water sampling from weekly to daily sampling, or even taking hourly samples for a 24-48h period, could reveal greater insights into the effectiveness of these constructed wetlands in treating different loads of pollutants. This is especially important in communities such as Silvertown that have regular time schedules for various activities e.g. washing clothes in the morning, bathing at night etc.

Along with an increase in the frequency of testing, including tests for biological oxygen demand, although they are more labour intensive and require more apparatus, would assist in gaining deeper insight into the quality of the wastewater in Silvertown, and the level of biological activity taking place inside of the wetland.

Overlaying the results obtained from the water quality tests with localised rain volumes could also yield insights into the pollutant removal efficiencies of these constructed wetlands. For example, correlating rainfall to the quality of water in and out of the wetlands, as well as the river water quality, could yield further insights into the extent of dilution of the water during rainfall. Regrettably, localised rain volumes were not available for Silvertown for the period of this study.

Regarding the design of the constructed wetlands, several improvements could be made in future design iterations. Within the design philosophy explored in Section 3.2.1, there is a compromise to be made between using materials and labour that are locally procurable and ensuring that the constructed wetland is designed and built in a way that maximises pollutant removal efficiency. In this study, priority was often given to locally-procurable materials, at the cost of the effectiveness of the system. A few particular aspects of the design could be improved. Firstly, the gravel size that was procured locally, was about twice the ideal size that would be required for optimal wastewater treatment (4-5cm stone size vs 2cm stone size). Secondly, waterproofing of both constructed wetlands proved to be an issue, and this can be improved by plastering the inside of the constructed wetlands and then adding the waterproofing bitumen paint. Lastly, in the course of this study it was

found that influent wastewater contained a high concentration of suspended solids and particulates from food. This can lead to the clogging of the constructed wetland and decrease overall pollutant removal efficiency. To address this, a settling chamber that can easily be cleaned out to remove settled solids and particulates could be design and implemented in future iterations.

6 References

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7 Appendix 1

Full results from water quality analyses.

Week	Date	COD Jukskei Upstream	COD CW 1 In	COD CW 1 Out	COD CW 2 In	COD CW 2 Out	COD Jukskei Downstream
1	16.09.2020	29	5090	5090	4855	5540	94
2	23.09.2020		5015	1930	3035	3795	
3	30.09.2020		3130	3355	3340	2760	78
4	08.10.2020		4370	2985	2970	2860	58
5	14.10.2020	23	4570	2890	3895	3760	70
6	22.10.2020	31	2940	2400	3065	2990	83
7	29.10.2020		5540	1970	3520	4945	62
8	05.11.2020		3600	3285	5200	2645	43
9	12.11.2020	34	1925	3727	3435	2527	62
10	19.11.2020	24	3807	3135	2790	3365	104
11	26.11.2020		4435	2790	3515	4027	46
12	03.12.2020		2820	3200	2415	3750	66
Week	Date	ORP Jukskei Upstream	ORP CW 1 In	ORP CW 1 Out	ORP CW 2 In	ORP CW 2 Out	ORP Jukskei Downstream
1	16.09.2020	29			-455.1	-404.4	-19.6
2	23.09.2020		-311.1	-323.9	-353.3	-331.9	28
3	30.09.2020	-16.6	-305.1	-366.6	-334.5	-308.7	-77.3
4	08.10.2020		-216.5	-326.9	-284.1	-337.7	-32.2
5	14.10.2020	5	-193.8	-308.6	-272.2	-322	-2.2
6	22.10.2020	200.1	-262.7	-325.7	-337.2	-361.8	28.9
7	29.10.2020		-213.1	-296.6	-285.3	-359.7	-68.5
8	05.11.2020		-147.9	-276.5	-258.3	-260.5	20.5
9	12.11.2020	193.0	-182.8	-305.6	-252.7	-281.9	-79.0
10	19.11.2020	-41.7	-168.2	-301.5	-289.3	-324.2	-122.1
11	26.11.2020		-62.3	-218.1	-363.8	-315.3	-105.8
12	03.12.2020		-183.4	-275.1	-307.3	-335.3	-99.2
Week	Date	EC Jukskei Upstream	EC CW 1 In	EC CW 1 Out	EC CW 2 In	EC CW 2 Out	EC Jukskei Downstream
1	16.09.2020				2356	4073	638
2	23.09.2020		2453	3515	2261	3204	610
3	30.09.2020	428	2996	3357	2427	2932	555
4	08.10.2020		3301	2949	1312	2704	575
5	14.10.2020	414	3246	3718	2529	3123	552
6	22.10.2020	553	3964	2569	3022	3530	623
7	29.10.2020		3528	2492	3284	4744	637
8	05.11.2020		3208	3314	5482	3328	283
9	12.11.2020	286	1667	4500	1131	2461	422
10	19.11.2020	419	2445	3729	1660	2994	540
11	26.11.2020		1672	2419	2634	2795	551
12	03.12.2020		1887	3505	1656	3335	521

Week	Date	pH Jukskei Upstream	pH CW 1 In	pH CW 1 Out	pH CW 2 In	pH CW 2 Out	pH Jukskei Downstream
1	**16.09.2020**				7.93	8.35	7.78
2	**23.09.2020**		8.50	7.92	8.00	8.06	7.70
3	**30.09.2020**	7.89	8.10	8.24	7.72	8.38	7.57
4	**08.10.2020**		7.09	8.14	8.23	8.72	7.56
5	**14.10.2020**	7.89	6.92	8.39	9.16	8.80	7.56
6	**22.10.2020**	7.91	7.44	7.72	8.58	8.41	7.80
7	**29.10.2020**		6.58	7.91	7.66	8.15	7.71
8	**05.11.2020**		7.12	7.91	8.77	7.84	7.65
9	**12.11.2020**	7.81	7.56	7.85	7.66	8.20	7.58
10	**19.11.2020**	7.97	6.54	7.55	7.97	8.16	7.74
11	**26.11.2020**		5.92	7.71	8.66	8.57	7.61
12	**03.12.2020**		7.15	7.64	8.51	8.73	7.52

Week	Date	TC Jukskei Upstream	TC CW 1 In	TC CW 1 Out	TC CW 2 In	TC CW 2 Out	TC Jukskei Downstream
1	**16.09.2020**	159295000	2.42E+11	6.2701E+10	1.973E+10	954000000	35214666667
2	**23.09.2020**		1.36E+09	6980000000	8.233E+09	1344000000	15863333333
3	**30.09.2020**	105288888.9		2546666667	1.986E+11	1548666667	15676666667
4	**08.10.2020**		3.466E+11	3740000000	2.42E+11	2.8939E+10	20720000000
5	**14.10.2020**	171066666.7	1.26E+09	2.8037E+10	6.76E+10	8450000000	23560000000
6	**22.10.2020**	552800000	2.42E+11	3.1237E+10	1.54E+11	1.8107E+10	81280000000
7	**29.10.2020**			9773333333		2130000000	17890000000
8	**05.11.2020**						
9	**12.11.2020**	345666666.7		3.3247E+10	4.794E+10	3428888889	37503333333
10	**19.11.2020**	393055555.6		1.0253E+10	4.011E+10	3790000000	27890000000
11	**26.11.2020**		6.619E+10	4.777E+10	2.159E+10	1.2099E+10	9642222222
12	**03.12.2020**		8.931E+10	1.7257E+10	4.138E+09		14582222222

Week	Date	E.coli Jukskei Upstream	E.coli CW 1 In	E.coli CW 1 Out	E.coli CW 2 In	E.coli CW 2 Out	E.coli Jukskei Downstream
1	**16.09.2020**	3055000	5229333333	25938666667	20000000	0	2703333333
2	**23.09.2020**		22910000000	785000000	436666666.7	176000000	5143333333
3	**30.09.2020**	5666666.667	355000000	200000000	236666666.7	0	1530000000
4	**08.10.2020**		400000000	0	620000000	864000000	685000000
5	**14.10.2020**	5800000	100000000	0	250000000	0	1463333333
6	**22.10.2020**	49000000	3150000000	270000000	1690000000	100000000	3665000000
7	**29.10.2020**		0	513333333.3	0	100000000	630000000
8	**05.11.2020**						
9	**12.11.2020**	18555555.56	34410000000	990000000	250000000	200000000	2922222222
10	**19.11.2020**	26866666.67	9156666667	33333333.33	166666666.7	100000000	2020000000
11	**26.11.2020**		12720000000	9687777778	1000000000	214444444.4	876666666.7
12	**03.12.2020**		36537777778	970000000	33333333.33	0	1560000000

Week	Date	Fluoride Jukskei Upstream	Fluoride CW 1 In	Fluoride CW 1 Out	Fluoride CW 2 In	Fluoride CW 2 Out	Fluoride Jukskei Downstream
1	16.09.2020			35.327	261.827	45.051	
2	23.09.2020		1.820	8.158	14.711		6.163
3	30.09.2020		9.490	14.515	11.837	9.329	
4	08.10.2020		18.557	9.144	54.878	9.676	34.576
5	14.10.2020		19.98	21.96	195.38	72.18	
6	22.10.2020	3.214					
7	29.10.2020		0.928	11.95	194.35		
8	05.11.2020						
9	12.11.2020		0.985	8.43	2.98	0.45	
10	19.11.2020		25.89	41.89	23.76	29.01	
11	26.11.2020		46.31	23.65	29.98		6.072
12	03.12.2020		15.72	19.01	20.96	79.29	
Week	Date	Chloride Jukskei Upstream	Chloride CW 1 In	Chloride CW 1 Out	Chloride CW 2 In	Chloride CW 2 Out	Chloride Jukskei Downstream
1	16.09.2020	32.892					43.801
2	23.09.2020		10.742	21.931	66.580	14.667	13.902
3	30.09.2020		67.199				
4	08.10.2020		14.895			17.233	
5	14.10.2020	24.082	366.13	181.61	82.64	140.50	38.526
6	22.10.2020	24.969	140.48	165.68	136.86	69.20	22.053
7	29.10.2020		32.031	163.15	0.00	0.00	
8	05.11.2020						
9	12.11.2020	0.424			7075.23	21.05	
10	19.11.2020	40.250	302.48	276.53	31.66	74.39	29.376
11	26.11.2020		310.09	84.62	55.95	144.99	147.295
12	03.12.2020			162.90	92.30		15.06
Week	Date	Nitrate Jukskei Upstream	Nitrate CW 1 In	Nitrate CW 1 Out	Nitrate CW 2 In	Nitrate CW 2 Out	Nitrate Jukskei Downstream
1	16.09.2020	44.211	286.635			118.763	35.462
2	23.09.2020						
3	30.09.2020	3.304					0.187
4	08.10.2020						19.485
5	14.10.2020	6.111			0.54	0.61	13.180
6	22.10.2020	54.99				27.42	3.372
7	29.10.2020			0.74		6.00	
8	05.11.2020						
9	12.11.2020		748.282				
10	19.11.2020	8.824					6.263
11	26.11.2020		1.80			6.27	3.069
12	03.12.2020		499.80		0.85	6.68	

Week	Date	Nitrite Jukskei Upstream	Nitrite CW 1 In	Nitrite CW 1 Out	Nitrite CW 2 In	Nitrite CW 2 Out	Nitrite Jukskei Downstream
1	16.09.2020	60.615				0.383	0.357
2	23.09.2020						
3	30.09.2020		274.003	113.492	230.858	193.322	31.245
4	08.10.2020		245.566	128.293	50.841	138.171	33.818
5	14.10.2020		41.79	10.80	3.14	11.88	1.22
6	22.10.2020	5.234				3.73	0.533
7	29.10.2020			4.42		10.39	
8	05.11.2020						
9	12.11.2020		900.726	532.50	15736.18	141.71	
10	19.11.2020						
11	26.11.2020		770.04				
12	03.12.2020					19.25	

Week	Date	Phosphate Jukskei Upstream	Phosphate CW 1 In	Phosphate CW 1 Out	Phosphate CW 2 In	Phosphate CW 2 Out	Phosphate Jukskei Downstream
1	16.09.2020		94.205	20.954	9.346	15.051	2.448
2	23.09.2020		1.425		3.498		4.468
3	30.09.2020		17.413	8.752		1.841	
4	08.10.2020		39.506	7.023	1.251	7.170	
5	14.10.2020						
6	22.10.2020						
7	29.10.2020		34.4055455	14.44653095	11.95246989		
8	05.11.2020						
9	12.11.2020		76.774	13.39	231.09	5.18	
10	19.11.2020		154.32	34.57		8.15	
11	26.11.2020		46.29		7.66		58.669
12	03.12.2020		72.22		3.44		

Week	Date	Sulfate Jukskei Upstream	Sulfate CW 1 In	Sulfate CW 1 Out	Sulfate CW 2 In	Sulfate CW 2 Out	Sulfate Jukskei Downstream
1	16.09.2020						
2	23.09.2020						
3	30.09.2020						
4	08.10.2020						
5	14.10.2020						
6	22.10.2020	32.543	178.13	76.56	78.25	73.66	
7	29.10.2020					77.63	
8	05.11.2020						
9	12.11.2020	66.291	267.422	151.85	164153.35	261.03	
10	19.11.2020	46.976	103.22	22.79	281.91	101.99	42.928
11	26.11.2020		17.11	330.14	419.86	116.03	73.717
12	03.12.2020		1.73	357.79	789.61	113.02	

Raw Data from Total Coliform and *E. coli* tests

	Week1 (16.09.2020)	Sample size(ul)	Big	Small	Big	Small	Total MPN	E MPN	Tcfu/100ml	Ecfu/100ml	Tcfu/100ml (With BDL)	Ecfu/ (With BDL)	Av. Tcfu/100ml	Av. Ecfu/100ml
1	Upstream	200	48	15	8	1	218.7	9.7	109350000	4850000	109350000	4850000		
2	Upstream	500	49	39	6	0	1046.2	6.3	209240000	1260000	209240000	1260000	159295000	3055000
3	Upstream (HPC)	500	49	48	49	48	>2419.6	>2419.6	#VALUE!	#VALUE!	BDL	BDL		
4	W1IN	1	49	47	39	20	2419.6	120.3	2.4196E+11	12030000000	2.4196E+11	12030000000		
5	W1IN	5	49	48	20	46	>2419.6	90.7	#VALUE!	1814000000	BDL	1814000000	2.4196E+11	5229333333
6	W1IN	5	49	48	21	45	>2419.6	92.2	#VALUE!	1844000000	BDL	1844000000		
7	W1OUT	1	49	34	7	8	770.1	770.1	77010000000	77010000000	77010000000	77010000000		
8	W1OUT	5	49	48	9	8	>2419.6	18.7	#VALUE!	374000000	BDL	374000000	62701000000	25938666667
9	W1OUT	5	49	47	7	13	2419.6	21.6	48392000000	432000000	48392000000	432000000		
10	W2IN	1	48	21	0	0	285.1	<1	28510000000	#VALUE!	28510000000	BDL		
11	W2IN	5	49	40	1	0	1119.9	1	22398000000	20000000	22398000000	20000000	19726666667	20000000
12	W2IN	5	49	43	0	0	413.6	<1	8272000000	#VALUE!	8272000000	BDL		
13	W2OUT	1	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
14	W2OUT	5	31	5	0	0	54.6	<1	1092000000	#VALUE!	1092000000	BDL	954000000	#DIV/0!
15	W2OUT	5	25	5	0	0	40.8	<1	816000000	#VALUE!	816000000	BDL		
16	Downstream	1	49	24	20	3	435.2	28.8	43520000000	2880000000	43520000000	2880000000		
17	Downstream	5	49	40	45	14	1119.9	152.9	22398000000	3058000000	22398000000	3058000000	35214666667	2703333333
18	Downstream	5	49	46	38	18	1986.3	108.6	39726000000	2172000000	39726000000	2172000000		
19	Blank	5	0	0	0	0	<1	<1	0	0	0	0	0	0

	Week2 (23.09.2020)	Sample size(ul)	Big	Small	Big	Small	Total MPN	E MPN	Tcfu/100ml	Ecfu/100ml	Tcfu/100ml (With BDL)	Ecfu/ (With BDL)	Av. Tcfu/100ml	Av. Ecfu/100ml
1	Upstream	*	*	*	*	*			#VALUE!	#VALUE!	BDL	BDL		
2	Upstream	*	*	*	*	*			#VALUE!	#VALUE!	BDL	BDL	#DIV/0!	#DIV/0!
3	Upstream	*	*	*	*	*			#VALUE!	#VALUE!	BDL	BDL		
4	W1IN	0.5	10	1	1	1	12.1	2	2420000000	400000000	2420000000	400000000		
5	W1IN	1	1	2	49	14	3	248.1	300000000	24810000000	300000000	24810000000	1360000000	22910000000
6	W1IN	1	0	0	49	24	<1	435.2	#VALUE!	43520000000	BDL	43520000000		
7	W1OUT	0.5	32	2	4	0	52.1	4.1	10420000000	820000000	10420000000	820000000		
8	W1OUT	1	44	3	7	0	102.2	7.5	10220000000	750000000	10220000000	750000000	6980000000	785000000
9	W1OUT	1	2	1	0	0	3	<1	300000000	#VALUE!	300000000	BDL		
10	W2IN	0.5	21	31	0	2	70.3	2	14060000000	400000000	14060000000	400000000		
11	W2IN	1	5	35	1	2	43.2	3	4320000000	300000000	4320000000	300000000	8233333333	436666666.7
12	W2IN	1	12	41	2	4	63.2	6.1	6320000000	610000000	6320000000	610000000		
13	W2OUT	1	4	2	1	1	6.2	2	620000000	200000000	620000000	200000000		
14	W2OUT	5	26	34	2	7	90.1	9.2	1802000000	184000000	1802000000	184000000	1344000000	176000000
15	W2OUT	5	29	23	4	3	80.5	7.2	1610000000	144000000	1610000000	144000000		
16	Downstream	0.5	40	20	23	1	127.4	31.3	25480000000	6260000000	25480000000	6260000000		
17	Downstream	1	34	25	31	4	104.7	52.9	10470000000	5290000000	10470000000	5290000000	15863333333	5143333333
18	Downstream	1	34	30	24	5	116.4	38.8	11640000000	3880000000	11640000000	3880000000		
19	Blank	1	0	0	0	0	<1	<1	0	0	0	0	0	0

	Week3 (30.09.2020)	Sample size(ul)	Big	Small	Big	Small	Total MPN	E MPN	Tcfu/100ml	Ecfu/100ml	Tcfu/100ml (With BDL)	Ecfu/ (With BDL)	Av. Tcfu/100ml	Av. Ecfu/100ml
1	Upstream	100	43	9	4	3	114.5	7.2	114500000	7200000	114500000	7200000		
2	Upstream	300	48	12	13	1	193.5	16	64500000	5333333.333	64500000	5333333.333	105288888.9	5666666.667
3	Upstream	300	49	23	11	1	410.6	13.4	136866666.7	4466666.667	136866666.7	4466666.667		
4	W1IN	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
5	W1IN	1	49	48	2	3	>2419.6	5.1	#VALUE!	510000000	BDL	510000000	#DIV/0!	355000000
6	W1IN	1	49	48	0	2	>2419.6	2	#VALUE!	200000000	BDL	200000000		
7	W1OUT	1	11	2	2	0	14.5	2	1450000000	200000000	1450000000	200000000		
8	W1OUT	1	24	5	0	0	38.8	<1	3880000000	#VALUE!	3880000000	BDL	2546666667	200000000
9	W1OUT	1	18	1	0	0	23.1	<1	2310000000	#VALUE!	2310000000	BDL		
10	W2IN	0.5	49	48	1	0	>2419.6	1	#VALUE!	200000000	BDL	200000000		
11	W2IN	1	49	48	4	0	>2419.6	4.1	#VALUE!	410000000	BDL	410000000	1.9863E+11	236666666.7
12	W2IN	1	49	46	1	0	1986.3	1	1.9863E+11	100000000	1.9863E+11	100000000		
13	W2OUT	1	13	0	0	0	14.8	<1	1480000000	#VALUE!	1480000000	BDL		
14	W2OUT	5	36	4	0	0	67.7	<1	1354000000	#VALUE!	1354000000	BDL	1548666667	#DIV/0!
15	W2OUT	5	41	5	0	0	90.6	<1	1812000000	#VALUE!	1812000000	BDL		
16	Downstream	0.5	36	7	8	1	73.8	9.7	14760000000	1940000000	14760000000	1940000000		
17	Downstream	1	45	17	11	1	167.4	13.4	16740000000	1340000000	16740000000	1340000000	15676666667	1530000000
18	Downstream	1	47	9	9	3	155.3	13.1	15530000000	1310000000	15530000000	1310000000		
19	Blank	1	0	0	0	0	<1	<1	0	0	0	0	0	0

	Week4 (08.10.2020)	Sample size(ul)	Big	Small	Big	Small	Total MPN	E MPN	Tcfu/100ml	Ecfu/100ml	Tcfu/100ml (With BDL)	Ecfu/ (With BDL)	Av. Tcfu/100ml	Av. Ecfu/100ml
1	Upstream	*	*	*	*	*			#VALUE!	#VALUE!	BDL	BDL		
2	Upstream	*	*	*	*	*			#VALUE!	#VALUE!	BDL	BDL	#DIV/0!	#DIV/0!
3	Upstream	*	*	*	*	*			#VALUE!	#VALUE!	BDL	BDL		
4	W1IN	0.5	49	45	0	2	1732.9	2	3.4658E+11	400000000	3.4658E+11	400000000		
5	W1IN	1	49	48	0	0	>2419.6	<1	#VALUE!	#VALUE!	BDL	BDL	3.4658E+11	400000000
6	W1IN	1	49	48	0	0	>2419.6	<1	#VALUE!	#VALUE!	BDL	BDL		
7	W1OUT	1	25	1	0	0	35	<1	3500000000	#VALUE!	3500000000	BDL		
8	W1OUT	1	20	4	0	0	30.1	<1	3010000000	#VALUE!	3010000000	BDL	3740000000	#DIV/0!
9	W1OUT	1	30	2	0	0	47.1	<1	4710000000	#VALUE!	4710000000	BDL		
10	W2IN	0.5	49	48	3	1	>2419.6	4.1	#VALUE!	820000000	BDL	820000000		
11	W2IN	1	49	48	5	1	>2419.6	6.3	#VALUE!	630000000	BDL	630000000	2.4196E+11	620000000
12	W2IN	1	49	47	3	1	2419.6	4.1	2.4196E+11	410000000	2.4196E+11	410000000		
13	W2OUT	1	49	20	10	1	344.8	12.1	34480000000	1210000000	34480000000	1210000000		
14	W2OUT	5	49	43	23	5	1413.6	36.8	28272000000	736000000	28272000000	736000000	28939333333	864000000
15	W2OUT	5	49	41	22	3	1203.3	32.3	24066000000	646000000	24066000000	646000000		
16	Downstream	0.5	42	7	0	0	101.7	<1	20340000000	#VALUE!	20340000000	BDL		
17	Downstream	1	48	12	6	0	193.5	6.3	19350000000	630000000	19350000000	630000000	20720000000	685000000
18	Downstream	1	49	12	6	1	224.7	7.4	22470000000	740000000	22470000000	740000000		
19	Blank	1	0	0	0	0	<1	<1	0	0	0	0	0	0

	Week5 (14.10.2020)	Sample size(ul)	Big	Small	Big	Small	Total MPN	E MPN	Tcfu/100ml	Ecfu/100ml	Tcfu/100ml (With BDL)	Ecfu/ (With BDL)	Av. Tcfu/100ml	Av. Ecfu/100ml
1	Upstream	100	44	20	2	3	167.9	5.1	167900000	5100000	167900000	5100000		
2	Upstream	300	49	28	16	4	547.5	23.8	182500000	7933333.333	182500000	7933333.333	171066666.7	5800000
3	Upstream	300	49	26	9	3	488.4	13.1	162800000	4366666.667	162800000	4366666.667		
4	W1IN	0.5	6	0	0	0	6.3	<1	1260000000	#VALUE!	1260000000	BDL		
5	W1IN	1	49	48	1	0	>2419.6	1	#VALUE!	100000000	BDL	100000000	1260000000	100000000
6	W1IN	1	49	48	0	1	>2419.6	1	#VALUE!	100000000	BDL	100000000		
7	W1OUT	0.5	41	5	0	0	90.6	<1	18120000000	#VALUE!	18120000000	BDL		
8	W1OUT	1	49	24	0	0	435.2	<1	43520000000	#VALUE!	43520000000	BDL	28036666667	#DIV/0!
9	W1OUT	1	49	12	0	0	224.7	<1	22470000000	#VALUE!	22470000000	BDL		
10	W2IN	0.5	49	27	2	0	517.2	2	1.0344E+11	400000000	1.0344E+11	400000000		
11	W2IN	1	49	31	0	1	648.8	1	64880000000	100000000	64880000000	100000000	67600000000	250000000
12	W2IN	1	49	20	0	0	344.8	<1	34480000000	#VALUE!	34480000000	BDL		
13	W2OUT	0.5	34	4	0	0	61.3	<1	12260000000	#VALUE!	12260000000	BDL		
14	W2OUT	1	30	5	0	0	54.6	<1	5460000000	#VALUE!	5460000000	BDL	8450000000	#DIV/0!
15	W2OUT	1	35	10	0	0	76.3	<1	7630000000	#VALUE!	7630000000	BDL		
16	Downstream	0.5	41	8	5	2	98.7	7.3	19740000000	1460000000	19740000000	1460000000		
17	Downstream	1	49	15	13	2	261.3	17.1	26130000000	1710000000	26130000000	1710000000	23560000000	1463333333
18	Downstream	1	49	14	11	0	248.1	12.2	24810000000	1220000000	24810000000	1220000000		
19	Blank	1	0	0	0	0	<1	<1	0	0	0	0	0	0

	Week6 (22.10.2020)	Sample size(ul)	Big	Small	Big	Small	Total MPN	E MPN	Tcfu/100ml	Ecfu/100ml	Tcfu/100ml (With BDL)	Ecfu/ (With BDL)	Av. Tcfu/100ml	Av. Ecfu/100ml
1	Upstream	100	48	32	32	6	478.6	59.1	478600000	59100000	478600000	59100000		
2	Upstream	300	49	46	44	8	1986.3	118.7	662100000	39566666.67	662100000	39566666.67	552800000	49000000
3	Upstream	300	49	44	47	7	1553.1	145	517700000	48333333.33	517700000	48333333.33		
4	W1IN	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
5	W1IN	1	49	47	23	4	2419.6	35.5	2.4196E+11	3550000000	2.4196E+11	3550000000	2.4196E+11	3150000000
6	W1IN	1	49	48	20	3	>2419.6	27.5	#VALUE!	2750000000	BDL	2750000000		
7	W1OUT	0.5	39	4	1	0	78.9	1	15780000000	200000000	15780000000	200000000		
8	W1OUT	1	49	26	3	0	488.4	3.1	48840000000	310000000	48840000000	310000000	31236666667	270000000
9	W1OUT	1	49	17	2	1	290.9	3	29090000000	300000000	29090000000	300000000		
10	W2IN	0.5	49	34	7	5	770.1	12.8	1.5402E+11	2560000000	1.5402E+11	2560000000		
11	W2IN	1	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL	1.5402E+11	1690000000
12	W2IN	1	49	48	3	5	>2419.6	8.2	#VALUE!	820000000	BDL	820000000		
13	W2OUT	0.5	26	3	0	0	39.9	<1	7980000000	#VALUE!	7980000000	BDL		
14	W2OUT	1	46	15	1	0	172.5	1	17250000000	100000000	17250000000	100000000	18106666667	100000000
15	W2OUT	1	49	17	0	0	290.9	<1	29090000000	#VALUE!	29090000000	BDL		
16	Downstream	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
17	Downstream	1	49	39	21	3	1046.2	30.5	1.0462E+11	3050000000	1.0462E+11	3050000000	81280000000	3665000000
18	Downstream	1	49	29	26	5	579.4	42.8	57940000000	4280000000	57940000000	4280000000		
19	Blank	1	0	0	0	0	<1	<1	0	0	0	0	0	0

	Week7 (29.10.2020)	Sample size(ul)	Big	Small	Big	Small	Total MPN	E MPN	Tcfu/100ml	Ecfu/100ml	Tcfu/100ml (With BDL)	Ecfu/ (With BDL)	Av. Tcfu/100ml	Av. Ecfu/100ml
1	Upstream	*	*	*	*	*			#VALUE!	#VALUE!	BDL	BDL		
2	Upstream	*	*	*	*	*			#VALUE!	#VALUE!	BDL	BDL	#DIV/0!	#DIV/0!
3	Upstream	*	*	*	*	*			#VALUE!	#VALUE!	BDL	BDL		
4	W1IN	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
5	W1IN	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL	#DIV/0!	#DIV/0!
6	W1IN	1	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
7	W1OUT	0.5	43	5	4	2	101.9	6.2	20380000000	1240000000	20380000000	1240000000		
8	W1OUT	0.5	18	2	1	0	24.3	1	4860000000	200000000	4860000000	200000000	9773333333	513333333.3
9	W1OUT	1	25	5	1	0	40.8	1	4080000000	100000000	4080000000	100000000		
10	W2IN	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
11	W2IN	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL	#DIV/0!	#DIV/0!
12	W2IN	1	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
13	W2OUT	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
14	W2OUT	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL	2130000000	100000000
15	W2OUT	1	16	2	1	0	21.3	1	2130000000	100000000	2130000000	100000000		
16	Downstream	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
17	Downstream	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL	17890000000	630000000
18	Downstream	1	48	10	6	0	178.9	6.3	17890000000	630000000	17890000000	630000000		
19	Blank	0.5	0	0	0	0	<1	<1	0	0	0	0	0	0

	Week8 (05.11.2020)	Sample size(ul)	Big	Small	Big	Small	Total MPN	E MPN	Tcfu/100ml	Ecfu/100ml	Tcfu/100ml (With BDL)	Ecfu/ (With BDL)	Av. Tcfu/100ml	Av. Ecfu/100ml
1	Upstream	*	*	*	*	*					BDL	BDL		
2	Upstream	*	*	*	*	*					BDL	BDL	#DIV/0!	#DIV/0!
3	Upstream	*	*	*	*	*					BDL	BDL		
4	W1IN	0.5	BDL	BDL	BDL	BDL					BDL	BDL		
5	W1IN	1	BDL	BDL	BDL	BDL					BDL	BDL	#DIV/0!	#DIV/0!
6	W1IN	3	BDL	BDL	BDL	BDL					BDL	BDL		
7	W1OUT	0.5	BDL	BDL	BDL	BDL					BDL	BDL		
8	W1OUT	1	BDL	BDL	BDL	BDL					BDL	BDL	#DIV/0!	#DIV/0!
9	W1OUT	3	BDL	BDL	BDL	BDL					BDL	BDL		
10	W2IN	0.5	BDL	BDL	BDL	BDL					BDL	BDL		
11	W2IN	1	BDL	BDL	BDL	BDL					BDL	BDL	#DIV/0!	#DIV/0!
12	W2IN	3	BDL	BDL	BDL	BDL					BDL	BDL		
13	W2OUT	0.5	BDL	BDL	BDL	BDL					BDL	BDL		
14	W2OUT	1	BDL	BDL	BDL	BDL					BDL	BDL	#DIV/0!	#DIV/0!
15	W2OUT	3	BDL	BDL	BDL	BDL					BDL	BDL		
16	Downstream	0.5	BDL	BDL	BDL	BDL					BDL	BDL		
17	Downstream	1	BDL	BDL	BDL	BDL					BDL	BDL	#DIV/0!	#DIV/0!
18	Downstream	3	BDL	BDL	BDL	BDL					BDL	BDL		
19	Blank	1	BDL	BDL	BDL	BDL					BDL	BDL	0	0

	Week9 (12.11.2020)	Sample size(ul)	Big	Small	Big	Small	Total MPN	E MPN	Tcfu/100ml	Ecfu/100ml	Tcfu/100ml (With BDL)	Ecfu/ (With BDL)	Av. Tcfu/100ml	Av. Ecfu/100ml
1	Upstream	100	49	20	16	1	344.8	20.1	344800000	20100000	344800000	20100000		
2	Upstream	100	49	21	16	1	365.4	20.1	365400000	20100000	365400000	20100000	345666666.7	18555555.56
3	Upstream	300	49	38	29	3	980.4	46.4	326800000	15466666.67	326800000	15466666.67		
4	W1IN	0.5	49	48	33	11	>2419.6	71.4	#VALUE!	14280000000	BDL	14280000000		
5	W1IN	1	49	48	49	26	>2419.6	488.4	#VALUE!	48840000000	BDL	48840000000	#DIV/0!	34410000000
6	W1IN	3	49	48	49	41	>2419.6	1203.3	#VALUE!	40110000000	BDL	40110000000		
7	W1OUT	0.5	49	13	7	1	235.9	8.5	47180000000	1700000000	47180000000	1700000000		
8	W1OUT	1	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL	33246666667	990000000
9	W1OUT	3	49	29	6	2	579.4	8.4	19313333333	280000000	19313333333	280000000		
10	W2IN	0.5	46	12	0	0	156.5	<1	31300000000	#VALUE!	31300000000	BDL		
11	W2IN	1	49	28	0	4	547.5	4	54750000000	400000000	54750000000	400000000	47937777778	250000000
12	W2IN	3	49	45	0	3	1732.9	3	57763333333	100000000	57763333333	100000000		
13	W2OUT	0.5	13	1	1	0	16	1	3200000000	200000000	3200000000	200000000		
14	W2OUT	1	27	5	0	0	45	<1	4500000000	#VALUE!	4500000000	BDL	3428888889	200000000
15	W2OUT	3	37	7	0	0	77.6	<1	2586666667	#VALUE!	2586666667	BDL		
16	Downstream	0.5	49	7	16	0	179.3	18.9	35860000000	3780000000	35860000000	3780000000		
17	Downstream	1	49	21	21	2	365.4	29.2	36540000000	2920000000	36540000000	2920000000	37503333333	2922222222
18	Downstream	3	49	41	33	6	1203.3	62	40110000000	2066666667	40110000000	2066666667		
19	Blank	1	0	0	0	0	<1	<1	0	0	0	0	0	0

	Week10 (19.11.2020)	Sample size(ul)	Big	Small	Big	Small	Total MPN	E MPN	Tcfu/100ml	Ecfu/100ml	Tcfu/100ml (With BDL)	Ecfu/ (With BDL)	Av. Tcfu/100ml	Av. Ecfu/100ml
1	Upstream	100	49	13	21	2	235.9	29.2	235900000	29200000	235900000	29200000		
2	Upstream	100	49	33	26	6	727	44.3	727000000	44300000	727000000	44300000	393055555.6	26866666.67
3	Upstream	300	49	31	16	2	648.8	21.3	216266666.7	7100000	216266666.7	7100000		
4	W1IN	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
5	W1IN	1	49	48	33	18	>2419.6	85.2	#VALUE!	8520000000	BDL	8520000000	#DIV/0!	9156666667
6	W1IN	3	49	48	43	44	>2419.6	293.8	#VALUE!	9793333333	BDL	9793333333		
7	W1OUT	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
8	W1OUT	1	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL	10253333333	33333333.33
9	W1OUT	3	49	18	1	1	307.6	1	10253333333	33333333.33	10253333333	33333333.33		
10	W2IN	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
11	W2IN	1	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL	40110000000	166666666.7
12	W2IN	3	49	41	0	5	1203.3	5	40110000000	166666666.7	40110000000	166666666.7		
13	W2OUT	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
14	W2OUT	1	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL	3790000000	100000000
15	W2OUT	3	42	11	2	1	113.7	3	3790000000	100000000	3790000000	100000000		
16	Downstream	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
17	Downstream	1	47	20	16	1	231	20.1	23100000000	2010000000	23100000000	2010000000	27890000000	2020000000
18	Downstream	3	49	38	32	7	980.4	60.9	32680000000	2030000000	32680000000	2030000000		
19	Blank	1	0	0	0	0	<1	<1	0	0	0	0	0	0

	Week11 (26.11.2020)	Sample size(ul)	Big	Small	Big	Small	Total MPN	E MPN	Tcfu/100ml	Ecfu/100ml	Tcfu/100ml (With BDL)	Ecfu/ (With BDL)	Av. Tcfu/100ml	Av. Ecfu/100ml
1	Upstream	*	*	*	*	*			#VALUE!	#VALUE!	BDL	BDL		
2	Upstream	*	*	*	*	*			#VALUE!	#VALUE!	BDL	BDL	#DIV/0!	#DIV/0!
3	Upstream	*	*	*	*	*			#VALUE!	#VALUE!	BDL	BDL		
4	W1IN	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
5	W1IN	1	49	27	37	9	517.2	82	51720000000	8200000000	51720000000	8200000000	66186666667	12720000000
6	W1IN	3	49	47	49	27	2419.6	517.2	80653333333	17240000000	80653333333	17240000000		
7	W1OUT	0.5	40	9	13	3	95.9	18.3	19180000000	3660000000	19180000000	3660000000		
8	W1OUT	1	49	34	46	11	770.1	151.5	77010000000	15150000000	77010000000	15150000000	47770000000	9687777778
9	W1OUT	3	49	43	49	18	1413.6	307.6	47120000000	10253333333	47120000000	10253333333		
10	W2IN	0.5	31	4	7	1	52.9	8.5	10580000000	1700000000	10580000000	1700000000		
11	W2IN	1	49	19	1	2	325.5	3	32550000000	300000000	32550000000	300000000	21585555556	1000000000
12	W2IN	3	49	31	0	0	648.8	<1	21626666667	#VALUE!	21626666667	BDL		
13	W2OUT	0.5	42	8	1	0	104.6	1	20920000000	200000000	20920000000	200000000		
14	W2OUT	1	34	7	3	1	67	4.1	6700000000	410000000	6700000000	410000000	12098888889	214444444.4
15	W2OUT	3	48	19	0	1	260.3	1	8676666667	33333333.33	8676666667	33333333.33		
16	Downstream	0.5	20	1	3	0	26.2	3.1	5240000000	620000000	5240000000	620000000		
17	Downstream	1	48	6	10	3	153.9	14.4	15390000000	1440000000	15390000000	1440000000	9642222222	876666666.7
18	Downstream	3	48	18	13	2	248.9	17.1	8296666667	570000000	8296666667	570000000		
19	Blank	1	0	0	0	0	<1	<1	0	0	0	0	0	0

	Week12 (03.12.2020)	Sample size(ul)	Big	Small	Big	Small	Total MPN	E MPN	Tcfu/100ml	Ecfu/100ml	Tcfu/100ml (With BDL)	Ecfu/ (With BDL)	Av. Tcfu/100ml	Av. Ecfu/100ml
1	Upstream	*	*	*	*	*			#VALUE!	#VALUE!	BDL	BDL		
2	Upstream	*	*	*	*	*			#VALUE!	#VALUE!	BDL	BDL	#DIV/0!	#DIV/0!
3	Upstream	*	*	*	*	*			#VALUE!	#VALUE!	BDL	BDL		
4	W1IN	0.5	49	48	49	24	>2419.6	435.2	#VALUE!	87040000000	BDL	87040000000		
5	W1IN	1	49	45	46	23	1732.9	225.4	1.7329E+11	22540000000	1.7329E+11	22540000000	89306666667	36537777778
6	W1IN	3	48	7	1	0	159.7	1	5323333333	33333333.33	5323333333	33333333.33		
7	W1OUT	0.5	23	6	0	0	38.3	<1	7660000000	#VALUE!	hg	BDL		
8	W1OUT	1	49	20	8	1	344.8	9.7	34480000000	970000000	34480000000	970000000	17256666667	970000000
9	W1OUT	3	1	0	0	0	1	<1	33333333.33	#VALUE!	33333333.33	BDL		
10	W2IN	0.5	1	0	0	0	1	<1	200000000	#VALUE!	200000000	BDL		
11	W2IN	1	27	4	0	0	43.5	<1	4350000000	#VALUE!	4350000000	BDL	4137777778	33333333.33
12	W2IN	3	49	13	1	0	235.9	1	7863333333	33333333.33	7863333333	33333333.33		
13	W2OUT	0.5	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
14	W2OUT	1	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL	#DIV/0!	#DIV/0!
15	W2OUT	3	0	0	0	0	<1	<1	#VALUE!	#VALUE!	BDL	BDL		
16	Downstream	0.5	35	9	9	1	74.3	10.9	14860000000	2180000000	14860000000	2180000000		
17	Downstream	1	49	14	14	1	248.1	17.3	24810000000	1730000000	24810000000	1730000000	14582222222	1560000000
18	Downstream	3	44	9	18	1	122.3	23.1	4076666667	770000000	4076666667	770000000		
19	Blank	1	0	0	0	0	<1	<1	0	0	0	0	0	0