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Technical feasibility of constructed wetlands to provide for improved environmental and cultural outcomes at Wairoa's wastewater treatment plant

Bachelorarbeit

Im Studiengang Umwelttechnik

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A	Surface	[m^2]
A	Number of arms	
A_{TK}	Surface trickling filter	[m^2]
AWF	All Weather Flow	
B	Filter bed profile	[m]
$B_{d,BOD,ZB}$	BOD load inflow per	[kg/d]
$B_{d,TKN,ZB}$	TKN load inflow	[kg/d]
$B_{R,BOD5}$	Volumetric BOD_5 loading	[kg/(m^3*d)]
BOD_5	Biochemical Oxygen Demand over 5 days	
C_{BOD5}	Concentration Biochemical Oxygen Demand	[g/m^3]
C_0	Influent concentration	[mg/l]
C_t	Effluent concentration	[mg/l]
COD	Chemical Oxygen Demand	
C_U	Uniformity coefficient	
d_{10}	Grain diameter at 10% passing	
d_{60}	Grain diameter at 60% passing	
DO	Dissolved Oxygen	
DSLO Class	Depth to a slowly permeable horizon	
DWF	Dry Weather Flow	
F	Face area	[m^2]
FIS	Floating Island Structure	
FTW	Free-Floating treatment Wetland	
Δh	Height difference	[m]
HSSF	Horizontal Subsurface Flow	
i	Hydraulic slope	[-]
I&I	Inflow and Infiltration	
k_{BSB}	Degeneration Invariable	[m/d]
k_f	Hydraulic Conductivity Coefficient	[m/d]
L	Length	[m]
ΔL	Filter length	[m]
n	Rotations per hour	[1/h]

Q	Volumetric flowrate	[m^3]
q_a	Surface load	[m/h]
$q_{A,d}$	Daily surface load	[m/d]
$q_{a,TK}$	Trickling filter surface load	[m/h]
Q_d	Daily volumetric flowrate	[m^3/d]
Q_{dW}	Dry Weather flow	[m^3/d]
Q_t	Daily dry weather volumetric flowrate	[m^3/d]
Q_{TK}	Total trickling filter volumetric inflow rate	[m^3]
RV_t	Recirculation	
S_k	Spray Power	[mm/arm]
TKN	Total Kjeldahl Nitrogen	
TN	Total Nitrogen	
TP	Total Phosphorus	
t_R	Hydraulic retention time	[h]
TSS	Total Suspended Solids	
u	Flow velocity	[m/d]
VF	Vertical flow	
V_{min}	Minimum Volume	
V_R	Reactor Volume	[m^3]
V_{sedi}	Volume sedimentation pond	[m^3]
$V_{TK,C}$	Reactor Volume based on carbon removal	[m^3]
$V_{TK,N}$	Reactor Volume based on Nitrogen removal	[m^3]
WWF	Wet Weather Flow	

1. Introduction

Wairoa's wastewater discharge consent expires in May 2019. Since April 2016 the Wairoa District Council has worked towards a solution of how an application for a further discharge consent might look like. The Wairoa Wastewater Stakeholder Group was formed to assist the Council to find an adequate option for the future of Wairoa's wastewater system. One of the most significant goals of this process was to gain cultural acceptance for a new developed future discharge option.

Cultural considerations about discharge changes are important for an application for a resource consent, as requirements are regulated by law in the Resource Management Act (RMA 1991), New Zealand Waste Strategy (2002), and the Local Government Act (LGA2002). Currently the treated wastewater is discharged into the Wairoa River. Wairoa's Tangata Whenua, the native Maori, have a strong spiritual connection to the Wairoa River and big concerns about wastewater entering it. In Maori beliefs water has a spiritual health, which is damaged when water meets waste. Wastewater restoring would only be possible by releasing it back to the earth (earth mother).

Stopping the discharge into the river and irrigating all wastewater on land is the favoured option for Tangata Whenua. However, this option is not the most practicable one for Wairoa, due to high costs and limited irrigation areas. Research and consultations with Wairoa's Stakeholder Group showed the significance of improving the rivers' health. The outcome is considered to be a package which includes wastewater infrastructure improvements, some wastewater irrigation and develop options to improve the overall health of the Wairoa River.

This thesis investigates wastewater infrastructure improvements to address cultural requirements and increase the wastewater effluent quality. The aim is to assess the current wastewater treatment system and develop an upgrade that satisfies both, treatment requirements and cultural values. Natural treatment systems such as wetlands are successfully applied all over the world and are known for being low cost and cultural related wastewater treatment systems in New Zealand. A major goal of this thesis is to prove if and which kind of natural system could be a practicable option for Wairoa. As the wastewater reticulation system is exposed to high inflow and infiltration of storm water, resulting in high fluctuations of the inflow volume, it must be proved if a natural system can be designed and operated in accordance to high flow variations.

2. Problem and objective

The aim of this thesis is to identify upgrade options for Wairoa's wastewater treatment plant with a natural treatment system, which addresses environmental improvements as well as cultural requirements. Therefore, some major questions, that must be answered before design suggestions can be made, arise.

Regarding to cultural values it needs to be clarified, which kind of changes Tangata Whenua demand for and what is required for cultural acceptance. Wairoa's population consists of a high percentage of Maori, who desperately aim for changing water discharge conditions. As their cultural values can be interpreted differently, depending on tribal affiliation and spiritual beliefs, a base for suitable water treatment in Maori beliefs needs to be found. Over the course of this thesis, indigenous views on water and water treatment will be reviewed and transmitted onto treatment requirements, compared with solutions other Councils have been found.

Besides cultural satisfaction, environmental improvements want to be achieved. Due to the age and condition of its reticulation system, Wairoa's treatment plant has issues with high inflow and infiltration (I&I) of storm water into its wastewater reticulation system. High fluctuations of the inflow volume occur. When designing a natural system for Wairoa, it must be designed and operated in accordance to high flow variations.

A significant problem for designing an alternative is a lack of data concerning the whole treatment and reticulation system. While not much data is available, data of inflow and effluent quality is insufficient. Consequently, conclusions about the current performance are doubtful. In addition, several key parameters that are required for designing a structure according to general guidelines, are not available or inaccurate. Furthermore, it is not clear, which quality standard the future discharge consent requires. At this stage, a design cannot be created by considering the Regional Council's claims. Wairoa District Council assumes, quality standards will be higher than the current ones.

3. Understanding the site

3.1. Wairoa's Wastewater treatment plant

3.1.1. Original Pond and Treatment Structure

Wairoa's wastewater treatment plant is a two-pond system. The first pond is operated as an aeration lagoon with a step-screen (< 5mm) located at its inlet. One mechanical aerator is deployed to ensure oxidation. The normal operation capacity of the aeration pond is 4,750 m^3. It can store 5,350 m^3 at its maximum. Influent is coarsely screened before entering the aerated lagoon, which has a surface area of approximately 2,120 m^2. Primary treated wastewater flows from the outlet of the aerated lagoon by gravity through an underground pipeline to the maturation pond which is the second pond with an approximate surface of 10,970 m^2 (Wairoa District Council, 2017a).

The capacity of the maturation pond is variable. It stores treated wastewater during times when no discharge occurs. Consequently, it builds up over the day and discharges into the Wairoa River during night. The maximum capacity is 24,130 m^3. Usually it is operated at an approximate volume of 18,250 m^3. The freeboard of the treatment plant ponds is about 300 mm for the aerated pond and 500 mm for the maturation pond.

The operating depth of the aeration lagoon is between 3.0 and 3.4 m, while the second pond is usually operated at a depth of 2.5 m (variates between 2.0 and 3.0 m).

	Oxidation pond	Maturation pond
Surface	2,120 m^2	10,970 m^2
Depth (approx.)	3,0 – 3.4 m	2.0 – 3.0 m
Operational volume	4,750 m^3	18,250 m^3
Maximum operational volume	5,350 m^3	24,130 m^3
Pond freeboard	0.3 m	0.5 m
Buffer capacity	636 m^3	5,485 m^3
Pre- treatment	5 mm step screen	-

Table 3-1 Design Data of Wairoa Wastewater Treatment Plant

The effluent is discharged through a gravity fed underwater pipe into the Wairoa River estuary with an outgoing tide. The discharge pipe is roughly opposite of the river mouth in a subtidal area. As discharges are not allowed to occur during daytime (6am to 6pm) and incoming tides, the treatment plant includes a buffer storage capacity of approximately 5,400 m^3 (mostly provided by the 500 mm operating freeboard of the maturation pond) to store wastewater. To ensure that the discharge conditions are met, an automated valve forces wastewater to surcharge within the treatment plant ponds up to the 5,400m^3 capacity.

In case of high water levels that could exceed the overflow weirs, the Wastewater Treatment Plant contains an emergency overflow system, which redirects excess wastewater to bypass pipelines. There are three overflow weirs, located at the most critical areas in case of an overflow (Appendix H). The first one can be found at the aeration pond's inlet, right before the step screen. The second one sits on the connection pipe between the two ponds and the last weir is located at the outlet of the maturation pond. The overflows feed directly into the discharge pipe downstream of the discharge meter.

Figure 3-1 presents an aerial photograph of the Wairoa Wastewater treatment plant with its key features. It shows the current operational status.

Figure 3-1 Structure of Wairoa Treatment Plant (own image recording)

3.1.2. Resource Consent Requirements

With the current Resource Consent, Hawke's Bay Regional Council permits Wairoa District Council to discharge treated sewer effluent from the sewage treatment plant into the Wairoa River estuary. The Resource Consent was permitted in accordance to Rule 11.4.1 of the Regional Plan (June 1999), and the provisions of the Resource Management Act 1991 (Resource Management Act, 1991).

In the following, the main operational restrictions are outlined.

The total daily discharge is restricted to 5,400 m^3/d. Effluent from the wastewater treatment plant shall only be discharged during periods of ebb tide, 30 minutes to six hours after high tide between 6.00 pm and 6.00 am.

The treatment plant's effluent discharge must not exceed the following standards:

COD	not greater than 220 mg/l
Total Ammonia	not greater than 36 mg/l
Suspended Solids	not greater than 87 mg/l

Once a month, Wairoa District Council must provide representative composite samples for the following analytes:

pH

COD

Total Ammonia

Suspended Solids

Conductivity

Enterococci

E. coli

The current Resource Consent expires on 31st of May 2019. Therefore, Wairoa District Council is aiming to upgrade the wastewater treatment plant to address public concerns about the public and environmental health regarding to sewer effluent entering the River (LEI, 2017).

With the upcoming Resource Consent, Wairoa District Council is considering to discharge 24 hours per day to maintain the treatment plant at a continuous level and operate the discharge at a lower flow rate. Any assumptions about the new standards for the discharge quality cannot be made at this stage, but it is assumed that the new standards will be on a higher level than the current standards.

3.2. Environment

3.2.1. Landscape

Wairoa's wastewater treatment plant is located on Whakamahi Road, Wairoa, Hawke's Bay. As seen in figure 3-2, it was built in a certain remoteness (red circle), approx. 2.5 km away from Wairoa's city centre.

Figure 3-2 Satellite picture of Wairoa (GoogleMaps)

It sits on a saddle of Pilot Hill, close to an estuary of the Wairoa River, where the river flows into the Pacific Ocean. Figure 3-3 shows a topographic aerial image of the setting. Data is based on the LIDAR survey from Hawkes Bay Regional Council (HBRC). Wairoa District Council converted the measured data via GIS (Geographic Information Software) into an aerial map.

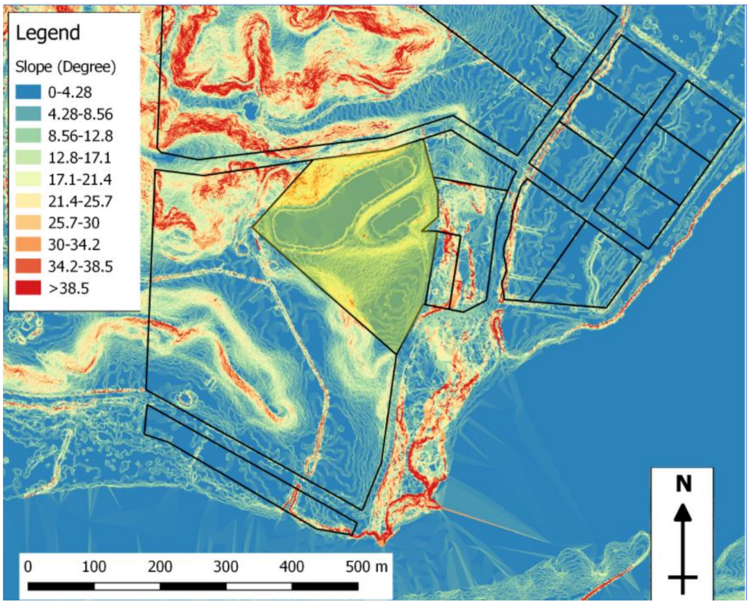

Figure 3-3 Topographic aerial view of Pilot Hill (GIS, Wairoa District Council)

The yellow marked area presents the property of the wastewater treatment facility. In the upper part, both ponds, aeration lagoon and maturation pond, are visible. The treatment plant is located on a saddle, surrounded by slopes. The northern terrain shows great slopes of >21.4 %, almost starting where the maturation pond is located. Gentle slopes and one great downhill slope can be found in the south western terrain of the area. The eastern terrain contains both, great and gentle slopes. Overall, the surrounding area is flat.

The New Zealand Land Cover Database (LCDB) shows, that the surrounding vegetation can be almost exclusively classified as High Producing Exotic Grassland. West of the treatment plant, there is only a small terrain of Broadleaved Indigenous Hardwoods (LCDB, 2015).
Two different soil types are presented on and around the treatment plant; Awamate silt loam and Gisborne sandy loam. As the main land of the treatment plant consists of Awamate silt loam, the following characteristics do not rely on Gisborne sandy loam.
The DSLO-class (depth to a slowly permeable horizon) describes the minimum and maximum depths to a horizon in metres, in which the permeability is less than 4 mm/hour (Newsome et al., 2008). It is divided into six classes. Awamate silt loam is classified as 2, with a minimum depth to a horizon of 0.9 m and a maximum of 1.5 m. The permeability of the soil, the rate that water moves through soil, is specified as moderate (Newsome et al., 2008).
These characteristics will be considered further on, while alternative treatment options are presented.

3.2.2. Land availability

On the treatment plant's property, there is an unused area of approximately 15,000 m^2 south of the ponds, as shown in figure 3-4. The section is overall flat, partly sloping.
Depending on type and design of natural treatment system, the area which could be used for construction is restricted to less than 15,000 m^2.

Figure 3-4 Available area south of the treatmend pond (GIS)

The southern part of the unused block, marked red in figure 3-4, offers approximately 7,300 m^2 available land, which is relatively flat. This is potentially more suitable for construction works than the yellow marked part, which has a great slope, shown in figure 3-3 of the topographical aerial of Pilot Hill.

Land outside the pond area is private land not owned by the Council and cannot be involved in any construction developments.

3.2.3. Wastewater management

Wairoa's wastewater reticulation system was built in 1948 and consists of 40 km gravity fed pipes in total. Thus, the majority (70%) of the pipe network is over 60 years old. However, improvements, extensions and replacements have been made since then. The reticulation system only receives domestic wastewater from 4,250 residents (Wairoa District Council 2017b), industrial wastewater is excluded.

In the early 1980s, the system was modified to a new gravity trunk sewer, which pumped sewage to the new treatment plant. Since then, four pump stations called Alexandra Park, North Clyde, Kopu Road and Fitzroy Street, collect water from each reticulation area and pump it up to the treatment facility. Alexandra Park, North Clyde and Kopu Road are essential lift stations, pumping the flow into a downgradient gravity sewer trunk main (see Fig 3-5). Fitzroy Street is a main collector pump station that pumps the combined flows through a pressurised pipe of approximately 560 m uphill to the wastewater treatment plant.

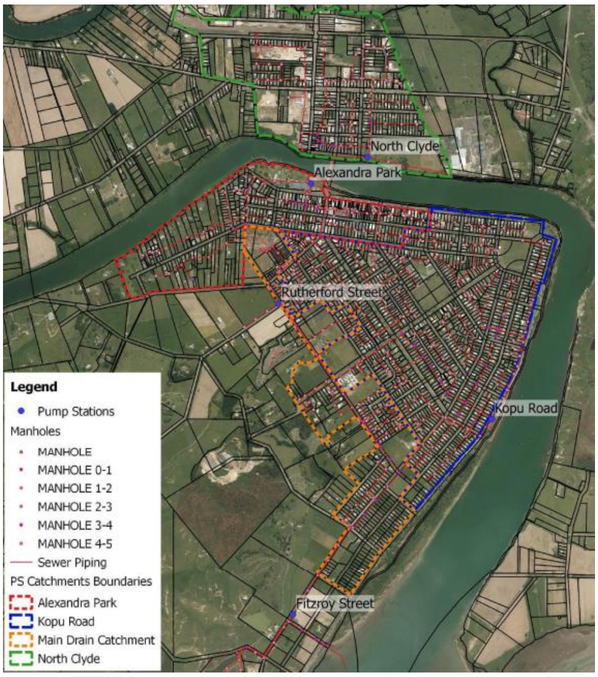

Figure 3-5 Wairoa wastewater reticulation system (LEI, 2017)

Reticulation currently allows significant stormwater and groundwater inflow into the system (LEI, 2017), which is typical for most of New Zealand's wastewater treatment plants.

Consequentially, high pump volumes and volume variations can occur (New Zealand Water & Waste Association, 2005).

Before water enters the first oxidation pond, which maintains a constant level through an overflow weir, water is screened by the step screen. From there, water flows into the maturation pond. The inflow of both ponds is approximately equal. The water level of the maturation pond varies, as levels rise over day when water builds up in the pond and fall during night-time, when the stored water is discharged. The pond has a buffer capacity of approximately 5,400 m^3 when the level is at its minimum point.
The wastewater treatment plant discharges treated effluent trough a gravity fed discharge pipe into the Wairoa River estuary, in a subtidal area approximately 150 m from the shoreline and approximately 800 m north-east of the river estuary which opens to the Pacific Ocean (EAM, 2012).

Figure 3-6 represents the water flow of Wairoa's wastewater treatment plant. Water inflow is illustrated as a dashed line, that leads from the collector pump station Fitzroy Street to the oxidation pond, into the maturation pond. Outflow is presented as a solid line, that leads from the maturation pond into the Wairoa river estuary (see figure3-6).

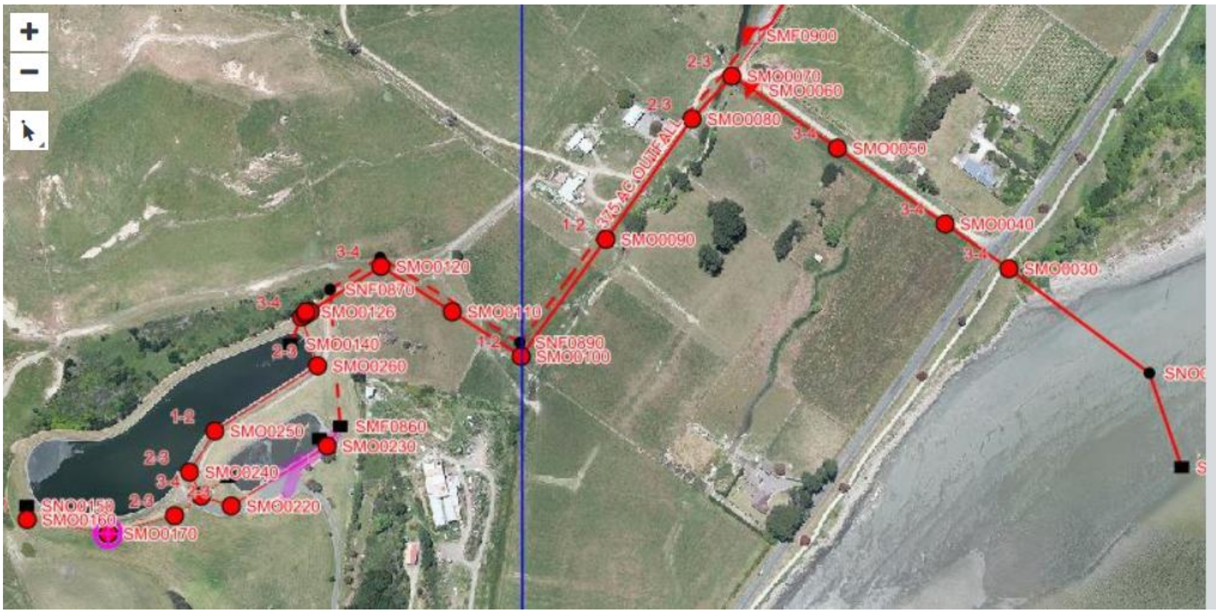

Figure 3-6 Water flow of Wairoa Wastewater Treatment Plant (GIS)

Environmental Assessment and Monitoring Ltd. surveyed the benthic effects of the Council's outfall discharge into the Wairoa River estuary and showed that the current wastewater outfall has no adverse effect on the benthic biota adjacent to the outfall (EAM, 2012).

4. Literature Review

4.1. Wastewater characteristics

4.1.1. Temperature

Many biological processes are temperature depended. The temperature is from great significance for the biological treatment. The activity of biological reactions increases with a rising temperature. Within this rising biological activity, the required oxygen demand increases as well. Moreover, the solubility of oxygen is temperature dependent. It is decreasing with increasing temperature. The combination of lower oxygen solubility and increasing biological activity can have a significant impact on the oxygen demand during higher temperatures (Metcalf & Eddy, 2014).

4.1.2. pH-Level

The pH-level of wastewater is significant for chemical and biological activity. Particularly biological organisms are sensitive for pH changes. For most organisms the critical range is between pH 6 and 9. For wastewater discharge, the pH-level should be between pH 6.5 and 8.5 (Metcalf & Eddy, 2014).

4.1.3. Nutrients

Nitrogen and phosphorus are known as nutrients or biostimulants. They are essential elements for the growth of microorganisms. Nitrogen has a complex chemistry due to its different states of oxidation. The most common and important forms of nitrogen in wastewater are Ammonia (NH_3), Ammonium (NH_4^+), Nitrogen gas (N_2), Nitrite ion (NO_2^-), and Nitrate (NO_3^-). The total amount of Nitrogen describes the total amount of Ammonia, Ammonium, Nitrite and Nitrate. Another common parameter for wastewater treatment is the Total Kjeldahl Nitrogen (TKN), which contains organic Nitrogen, Ammonia and Ammonium.

Phosphorus can be present in wastewater in two forms, either dissolved or particular. The soluble forms of Phosphorus include Orthophosphorus; reactive Phosphorus, which is directly available for biological metabolisation (Metcalf & Eddy, 2014).

4.1.4. Total Suspended Solids

Total Suspended Solids (TSS) are a common indicator for the performance assessment within the wastewater treatment process and determined by the suspended solids in mg per litre water. It is used as one of two universal test standards for regulatory control purpose. The TSS is usually determined by filtrating the wastewater (Metcalf &Eddy, 2014). The pollutants load can be associated with the load of suspended solids in the water (Kadlec & Wallace, 2009).

4.2. Parameters for wastewater calculations

4.2.1. Dissolved Oxygen Demand

Dissolved Oxygen Demand is an important parameter used for wastewater treatment technology. It is particularly significant for various pollutant removal mechanisms. Nitrification and aerobic decomposition require the presence of oxygen to occur. Dissolved oxygen is a common regulatory discharge parameter for treated wastewater into any surface water. Low levels of Dissolved Oxygen (DO) can adversely influence fish and other aquatic organisms. Oxygen intake into the wastewater can either occur due to biological oxygen production based on photosynthesis or through physical transfer into water. The maximum dissolved amount of oxygen in water depends on temperature and atmospheric pressure. Dissolved salts and biological activity can also influence the DO level in water. The DO level is reported in mg per litre (Kadlec & Wallace, 2009).

4.2.2. Biochemical Oxygen Demand (BOD)

The biochemical oxygen demand is the most common parameter to classify organic pollution of waste and/or surface water. The BOD is commonly considered as BOD_5, which determines the oxygen demand used by microorganisms for biochemical oxidation of organic matter within a period of five days and a temperature of 20 °C. The BOD reflects the amount of oxygen, which is required to biologically stabilize the organic matter of treated wastewater. It is further a relevant parameter for the sizing of treatment facilities and is often used to validate the efficiency of different treatment processes. The occurrence of nitrification can result in higher BOD readings. Therefore, the Carbonaceous Biochemical Oxygen Demand (CBOD) is considered to determine the true value of the oxygen demand to oxidize the organic matter. The value of BOD is expressed in milligram oxygen per litre. By Metcalf and Eddy, the theoretical value for the average BOD of wastewater is 60 g BOD_5 per capita per day or due to the concentration of 200 g/m^3 (Metcalf & Eddy, 2014). Values of concentrations vary globally, depending on the wastewater reticulation system.

4.2.3. Hydraulic parameters

When dimensioning wastewater systems, hydraulic parameters are highly significant, as it must be proved if a system can handle the expected hydraulic loads.

Flowrates are significant for any calculation regarding the hydraulic design for any wastewater treatment process (Metcalf & Eddy, 2014). Especially dry weather flow (DWF) and maximum flow are important flow parameters and the base for various hydraulic calculations. The German Association for Water and Waste (DWA) recommends to monitor the daily and annual flow variation to identify minimum and maximum flowrates. It is important to distinguish if the wastewater reticulation network is a mixed system which collects sewer and rain water or sewer exclusively. A mixed system will have bigger flow variations than a system without storm water contribution. Conclusions about the dry weather flow can be made according to the supplied potable water flow. Potable water consumers which do not contribute to the wastewater system (e.g. farms or companies with their own wastewater treatment) have to be excluded for any conclusion (DWA, 2003).

When designing wastewater infrastructure, future development of the flowrate should be considered. Infiltration of storm and surface water or increasing/decreasing population could potentially affect future flowrates (Metcalf & Eddy, 2014).

Surface load

The surface load describes the relation between flow rate and a specific of a reactor surface. The surface load is calculated as the quotient of flowrate and surface area (DWA, 2016).

$$q_A = \frac{Q}{A} \qquad (4.1)$$

Hydraulic retention time

The hydraulic retention time depends on the reactor volume and the inflow rate. It is calculated as the quotient of flowrate and reactor volume (DWA, 2016).

$$t_R = \frac{Q_d}{V_R} \qquad (4.2)$$

4.3. Maori worldview and its connection to water

4.3.1. Relevance of indigenous perspective

New Zealand values the indigenous perspectives of the Tangata whenua. The government policy and legislation support Maori values and recognise the importance of cultural respect, especially in terms of environmental management. First mentioned in the Treaty of Waitangi, signed in 1840 between The Queen of England and Maori people, environmental management is still relevant and therefore regulated by law in the Resource Management Act (RMA 1991), New Zealand Waste Strategy (2002), and the Local Government Act (LGA2002) (Morgan, 2006).

The Resource Management Act introduces the Maori concept of guardianship (Kaitiakitanga) and specifies several requirements about managing the use, development, and protection of natural and physical resources (RMA, 1991).

The Local Government Act mentions the importance of regional and territorial authorities when promoting social, economic, environmental well-being of their communities and approaches to a sustainable development (Local Government Act, 2002).

The New Zealand Waste Strategy points out, that the Tangata Whenua have a unique view on waste management with the ability to affect the process of wastewater treatment in New Zealand, as Maoris must be allowed to input directly into standards, guidelines, etc. and to consult in wastewater management and waste minimisation processes (Ministry for the Environment, 2002).

Despite governmental legislation, the Waitangi Tribunal received and receives several claims concerning the pollution of waters and wastewater effluents (Hughes, 1986; Morgan, 2006), which shows the cultural significance of the tribal waters to Tangata whenua. In practice, issues between engineering on one hand and Maori beliefs on the other, water management and treatment often cause disagreements and requires extremely sensitive handling of the traditional values and indigenous spirituality (Morgan, 2006).

4.3.2. Water's role in creation of earth

Maoris have always seen water as a treasure (taonga) (Morgan, 2006). To understand the significant role of water, it is necessary to look at their belief of the creation of earth. Several scientists who research Maori history point out, that the belief of water as a treasure goes back to the creation of the world, when the sky father (Ranginui) and the earth mother (Papatuanuku), joined together at the hip, were forced apart by their children. When the parents separated, the children spread around the several created realms between sky and earth, which are land, oceans, fresh water rivers and lakes, and the air space. The parents suffered from being separated, and that's how rain arised. Rain is meant to be the tears of the sky father, while well-spring water is to be considered as the "weeping of papa" ("papa" for Papatuanuku). Consequently, water from rainfall and springwater is sacred for Maori, and only suitable for human use after it has travelled over the earth mother to become profane. The earth mother, as the life provider to all living things, is recognised to derive life through the waters in her womb. Therefore, water as the "life giving essence" must be pure and genuine to provide life (Morgan, 2006; Ihaka, Awatere & Harrison, 2000).

According to statements by Morgan, Tangata whenua believe, that every living being, also water, and water bodies like rivers, lakes and swamps, has its own mauri, which is meant to be a binding force between the spiritual and physical being. Depending on the circumstances that influence a living being at a time, the nature of a mauri can vary from strong or weak to exhausted (Morgan, 2006).

4.3.3. Water categorisation and usage

The waters' mauri is one of the most important characteristics that needs to be considered when developing treatment systems. Due to the indigenous view, not every water is suitable for human use.
Water can be categorised by its physical and spiritual health and its geographical location.

Wai Ora is meant to be the purest form of water, not tainted physically or spiritually. Also called the life-giving water, it is used for bathing, healing and blessings. When it meets humans, the water becomes Wai Maori (Morgan, 2011). Wai Maori is clean and profane water, which is suitable for most uses like drinking and bathing. It also includes freshwaters that contain any food sources like eels, fish or flax (Love, 1990). Wai Tapu is sacred water due to its location or relationship to other sacred entities. Wai Tai is salt water, and stands for tidal and coastal water (Morgan, 2011). Water that has been exposed to pollution and can negatively affect other water sources, is called Wai Kino. It is also described as dangerous water. Wai mate categorises contaminated and polluted water, which is completely exhausted of its mauri. Waters with an exhausted mauri, must return to the earth mother to regain new life (Morgan, 2006; Douglas, 1984). Wai mate cannot be used for any consumption or other purposes and shouldn't be mixed with clean water, as it is seen at abhorrent. Consequently, Maori have a cultural abhorrence to the direct discharge of human wastewater to natural water, irrelevant of its level of treatment (Morgan, 2011; Bradley, 2015).

Figure 4-1 Natural Water Cycle (Ministry for the Environment, 2003)

4.3.4. Practical significance

As approximately 58% of Wairoa District's population identify themselves as "having Maori descent", water management is a significant issue (Statistics New Zealand, 2013). Due to the importance of water, the Tangata whenua would like to see it protected from pollution and damage. As mentioned before, Maoris have a guardianship role over natural resources, which is also part of the New Zealand legislation. In developing wastewater solutions, all participants must have regard for the principles of the Treaty of Waitangi, which means generally consultation with the local Maoris and showing them beneficial outcomes for the environment and culture. The traditional relationship that Tangata whenua have with their land, water, and sacred places needs to be considered when making decisions (Bradley, 2015).

Furthermore, working with Tangata whenua requires a sensitive approach in relation to cultural values and spirituality. It is necessary to respect their relationship to nature and human resources. Maoris also prefer face-to-face communication and personal relationships, than indirect contact. Understanding their environmental concepts and providing sustainable opportunities related to indigenous perspectives, provide an essential base to find common solutions (Bradley, 2015).

Ultimately, the question, which wastewater treatment system could meet cultural requirements arises. As stated above, one major key point for designing an alternative system must be respected; water must pass through the earth/earth mother Papatuanuku to be restored.

To satisfy cultural demands, the Ministry for the Environment suggests imitating the natural water cycle (Figure 4-1) when treating wastewater. Therefore, wetland systems are proposed to be used, as they are accepted by many maori tribes. As water is treated via “soil treatment”, a contact between water and earth is established (Ministry for the Environment, 2003).

In the following chapter of this thesis, the focus will be placed on wetlands as natural wastewater treatment.

4.4. Wetlands

Wetlands are land areas that are wet during part or all times of the year. The main characteristics of natural wetlands are the absence of plants that cannot grow in saturated media and the steady change of chemical, physical and biological soil properties during flooding. According to Kadlec and Wallace, wetlands are known to be one of the most biologically productive ecosystems on the planet (Kadlec & Wallace, 2009).

Wetlands have been recognized to have a high pollutant removal efficiency due to high biological activity, easy operation and maintenance and low energy requirements. From an ecological view, scientists describe the high rates of water recycling and potential to provide a wildlife habitat as remarkable (Kadlec & Wallace, 2009; Headley & Tanner, 2008).

In recent decades, wetlands gained more attention as an alternative wastewater treatment technology. Artificially built wetlands are commonly known as constructed wetlands or treatment wetlands and use natural occurring interactions between water, plants, microorganisms, soils and the atmosphere to remove contaminants from polluted water. Low building and operation costs and relatively easy maintenance are main advantages (Geller & Höner, 2003; Kadlec & Wallace, 2009; Headley & Tanner, 2006).

As progress reports showed in the past, wetlands are most suitable for smaller rural communities, according to required treatment surface area (Geller & Höner, 2003; Kadlec & Wallace, 2009).

In New Zealand, wetlands were initially developed to treat meat-processing water. From 1994 to the present, the National Institute for Water and Air (NIWA) investigated into research about horizontal subsurface flow wetlands (Kadlec & Wallace, 2009). One of the main drivers for the growth of wetland technology is the low investment and operating costs and the consideration of Maoris cultural and spiritual values (Kadlec & Wallace, 2009; Tanner et. al, 2006).

4.4.1. Types of wetlands

Treatment wetlands can be separated in two groups; surface flow and subsurface flow wetlands. Subsurface flow wetlands include horizontal and vertical flow wetlands, which are most commonly used.

The following chapter illustrates in the literature defined wetlands, as well as rock filters as natural wastewater treatment application.

4.4.1.1. Horizontal subsurface flow wetlands

Horizontal subsurface flow (HSSF) wetlands are typical sand or gravel beds, planted with wetland vegetation. The water flows below the surface from the inlet through the media in and around the roots and rhizomes of the plants, to the outlet in horizontal direction. HSSF wetlands are commonly used for wastewater treatment of smaller communities with lower flowrates and/or as secondary treatment process. Figure 4-2 shows a scheme of an HSSF wetland and its typical components (Kadlec & Wallace, 2009).

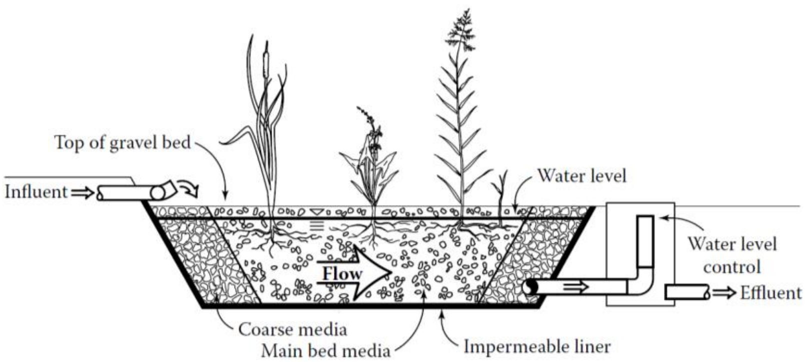

Figure 4-2 HSSF wetland schematic (Kadlec & Wallace, 2009)

Operation

For distribution of wastewater, water is induced via influent into a coarse media before it travels through the main bed media (filter material). Appropriate material for a distribution structure can be rough gravel, which supports an easy flow distribution. Account should be taken of particle size, as particle size of coarse media should be significantly greater than of the main bed media. An extra layer of material with a particle size smaller than distribution structure but greater than filter material can be considered to avoid shortcut flows into the main bed media. By traveling through the main filter material, water is exposed to metabolic processes. The principal biological treatment occurs. Therefore, hydraulic characteristics and adequate applied materials are critical for successful treatment (Geller & Höner, 2003).

A consistent flow in combination of evenly distributed water is required. The hydraulic

retention time is significant and must be sufficient for biological metabolism. Geller and Höner recommend a filter bed depth not greater than 500 mm, as deeper filter sizes could cause insufficient oxygen transfer in deeper areas (Geller & Höner, 2003).
The outlet structure is similar to the inlet. Rough gravel and a drainage pipe installed in lower parts of the system support an adequate runoff. Here again, the outlet structure should contain an extra layer of gravel with a particle size greater than filter material and smaller than the outlet channel to avoid washout of filter material.

According to Geller and Höner, 2003, the outlet structure should be moderately sloping and have the ability to alter the water level. To avoid overflows during high inflows, the wetland should contain a freeboard of 200-300 mm (Geller & Höner, 2003).

Assessment

In the literature, two different ways of designing HSSF wetlands can be found; according to hydraulic requirements or to degradation kinetics.
Hydraulic designs of HSSF wetlands are based on the required face area and the length of the filter bed. The entire face area should be as large as possible. Thus, the longest side of the filter bed is usually chosen as intake with several inflow pipes, where appropriate, for consistent water distribution. The length of the wetland depends on the filter material. Sand bed filters for example can have a rough estimated length of 3-5 m. Filter length and height difference between in- and outlet are required to calculate the hydraulic slope.

According to the equation of Darcy, the filter profile is calculated as follows (Geller & Höner, 2003).

$$F = \frac{Q}{k_f * I} \quad (4.3)$$

$$i = \frac{\Delta h}{\Delta L} \quad (4.4)$$

where

i	=	hydraulic slope	[-]
F	=	required face area	[m^2]
Q	=	maximum volumetric flow rate	[m^3/d]
k_f	=	hydraulic conductivity coefficient	[m/d]

Δh	=	height difference between water inlet and outlet	[m]
ΔL	=	filter length	[m]

By designing HSSF wetlands according to degeneration kinetics, the required area to metabolite a certain amount of biological mass load is essential. Concentrations of influent and effluent and the average daily feed volume are critical factors, which affect the required area.

The degeneration invariable k_{BOD} is based on empiric investigations of different filter beds and is typically between 0.06 and 0.1 m/d. The required filter surface is calculated as followed (Geller & Höner, 2003).

$$A = B * L = \frac{Q*(lnC_0 - lnC_t)}{k_{BOD}} \qquad (4.5)$$

where

A	=	required filter surface [m^2]
Q	=	volumetric flow rate [m^3/d]
C_0	=	influent concentration [mg/l]
C_t	=	required effluent concentration [mg/l]
k_{BOD}	=	degeneration invariable [m/d]
B	=	filter bed profile [m]
L	=	filter bed length [m]

According to ATV Worksheet A 262, filter bed surface-calculation can be simplified and achieve similar results by using the following references (DWA, 2006).

The overall filter surface should be considered as minimum	≥ 5 m^2/ capita
Minimum filter surface	≥ 20 m^2
COD surface load	≥ 16 g/m^2
Hydraulic surface load	≥ 40 mm/d = (40 l/(m^2/d)

4.4.1.2. Vertical flow wetlands

Vertical flow (VF) wetlands are constructed sand or gravel filters planted with various wetland plants. Through the influent, water is spread on the filter surface and infiltrates through the filter media in vertical direction before it discharges through underdrain pipes. VF wetlands are known for high removal rates of organics and suspended solids. Scientists like Kadlec and Wallace, as well as Geller and Höner, describe that VF wetlands provide a great nitrification potential. The overall nitrogen removal is based on the high oxygen transfer through all layers of the filter (Kadlec & Wallace, 2009; Geller & Höner, 2003). The required total area of a VF compared to a HSSF wetland is smaller, with the same performance (Geller & Höner, 2003).

Operation

Vertical flow wetlands can be operated in several ways as intermittent down flow, unsaturated down flow, saturated down-/up flow or tidal flow.

Intermittent down flow wetlands are pulse load operated. For short time periods, the filter bed surface is flooded before water soaks through the media. Geller and Höner speak of a "good" oxygen transfer through all layers of the filter bed, which intermittent down flow wetlands provide (Geller & Höner, 2003).

Unsaturated down flow operations irrigate wastewater on filter beds. Water can be distributed over the filter media in several ways, such as underground irrigation pipes, above-ground irrigation pipes or sprinkler irrigation systems. The irrigated water soaks into a sand or gravel bed and travels in vertical direction through the media. These filters can be operated in single pass mode or as a recirculation system where water passes through the media multiple times.

Saturated down or up flow filters provide a continuous saturated flow through the filter media. Down flow options can be either aerated or unaerated. Aerated down flow treatment is known for a great potential of ammonia removal. Up flow is commonly applied where daylighting water must be of high quality (Kadlec & Wallace, 2009).

Tidal flow operated wetlands provide a cycle of filling and draining. Influent and effluent are constructed on the bottom of the filter bed. Wastewater is pumped from the bottom of the filter bed until the filter surface is flooded. It stays flooded for a certain time to provide contact time between wastewater and filter material. Microorganisms, which are growing on the filter material, metabolite the organic pollutants of the wastewater during flooding. The continuous change of flooding and draining creates various redox conditions. Therefore, tidal wetlands are known for good nitrification and denitrification potential (Kadlec & Wallace, 2009; Geller & Höner, 2003).

The most applied option is intermittent down flow operated as single pass with pulse load operated VF, where the filter surface is flooded. This filter bed consists of layers of porous media to provide a smooth runoff. The bottom contains drainage pipes surrounded by coarse media. Drainage and contact time with the filter depends on filter size and media material and can vary from less than ten minutes to several hours (Kadlec & Wallace, 2009; Geller & Höner, 2003). Figure 4-3 shows a design of a VF wetland with design typical features.

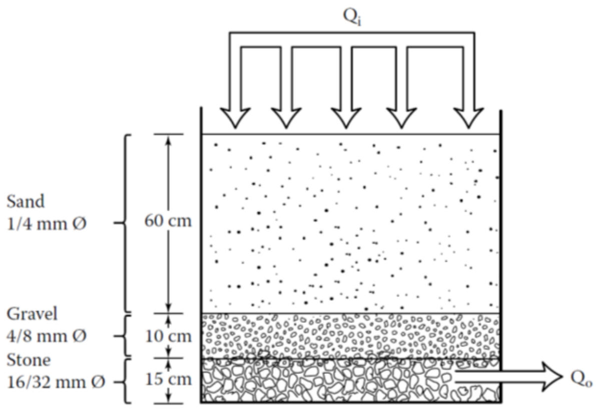

Figure 4-3 VF wetland schematic with design features (Kadlec & Wallace, 2009)

The oxygen transfer occurs mainly during the period of drainage when the draining water sucks air from the surface down in the lower layers of the filter. The influent should be pre-treated to remove solids out of the water and avoid clogging of the filter material (Kadlec & Wallace, 2009; Geller & Höner, 2003).

Assessment

The major parameter that is used for calculating the assessment is the filter surface area, which is required to degenerate the inflow mass load. The hydraulic load is not that important, as VF wetlands can be viewed as hydraulically overloaded during flooding, which occurs because of pulse rate operation (Geller & Höner, 2003).
According to ATV Worksheet A 262, the filter bed surface calculation can be generalised after the following reference values (DWA, 2006).

The overall filter surface should be considered as minimum	≥ 4 m^2/ capita
Minimum filter depth	≥ 0.5 m
Minimum flow rate	≥ 6 l/(m^2*min)

A more accurate calculation would be provided by assessing the real mass load. The surface should be calculated according to the mass load as this assessment can highlight potential colmatation and avoid the blockage of the filter material. DWA recommends assuming the following parameters to avoid colmatation:

Maximum COD surface load for all year operation	20 g/(m^2*d)
Maximum inflow concentration of filtratable substances	10 mg/l
Maximum surface load of filtratable substances	5 g/(m^2/d)
Maximum hydraulic load during summer	80 l/(m^2*d)
during winter	110 l/(m^2*d)

If no or only insufficient data is available, the COD load can be calculated according to DWA ATV A 262, as 75 g COD per capita after pre-treatment or as 50 g COD per capita, if the pre-treatment capacity is greater than 1 m^3 per capita (Geller & Höner, 2003).

4.4.1.3. Rock Filters

Rock filters can be applied in a wide range of designs. The classification depends on the type of operation. Rock filters can be constructed as submerged porous rock beds, either as vertical flow or horizontal subsurface flow. If a rock filter is operated as submerged vertical flow or horizontal flow, it can be classified as a VF or HSSF wetland. The major difference is the filter material used as a filter bed (Crites et. al, 2014). In New Zealand rock filters are added successfully to existing wastewater treatment plants (NZWWA, 2005). They are build as permeable embankments across treatment ponds, mainly to increase suspended solids removal, nitrogen removal and reduce short circulation. This application can be referred as interpond rock filter. To improve the nitrification potential the rock filter bed could include extra aeration (Crites et. al, 2014; NZWWA, 2005).

In the literature it can be found, that rock filters have been designed using a wide range of parameters, due to design variations. The critical factor when designing rock filters is the hydraulic loading rate. Rates less than 0.3 m^3/m^3*d have shown the best results for a vertical flow upstream operated rock filter with a rock size of 0.08 to 0.2 m and a depth of 2 m.

Systems listed in the literature range from small scale systems with a design inflow of 375 m^3/d to 3,300 m^3/d. All systems have a lower surface loading rate (<80 mm/d) in common, only the Paeroa rock filter operates with a flow rate greater than 80mm/d (91 mm/d) (Crites et. al, 2014).

Furthermore, rock filters can be applied as attached grow medium, used in a trickling filter (Metcalf & Eddy, 2014).
In New Zealand trickling filters are used to upgrade wastewater treatment in context of cultural considerations (Napier City Council, 2018; Gisborne District Council, 2016). Trickling

filters are fixed- bed reactors filled with an attached grow media, such as plastic or rock. The water is distributed through rotation sprinkler arms (Metcalf and Eddy, 2014). Aggregate size ranges from 40 mm to 80 mm grain size, it is recommended by DWA ATV-DVWK A 281, to introduce a layer on the bottom of the filter, with a grain size of 80 mm – 150 mm, usually applied to stop washout of filter material.

The biological treatment occurs through microorganisms growing on the filter material, which is the same biological treatment mechanism that occurs in wetlands (DWA, 2001; Kadlec & Wallace, 2009). In comparison to wetlands, trickling filters can handle higher hydraulic loading rates (Metcalf & Eddy, 2014; Kadlec & Wallace, 2009).

According to ATV-DVWK-A 281 trickling filters are designed based on BOD_5 volumetric loading rate and total Kjeldahl nitrogen (TKN) volumetric loading rate (DWA, 2001).

The reactor volume is calculated according to equation 4.6

$$V_{TK,C} = \frac{B_{d,BOD,ZB}}{B_{R,BOD}} \quad [m^3] \quad (4.6)$$

Where

$B_{d,BOD,ZB}$	=	BOD_5 load inflow	[kg/d]
$B_{R,BOD}$	=	Volumetric BOD_5 loading	[kg/(m³*d)]

Under consideration of nitrification, the reactor volume for nitrification is calculated as follows.

$$V_{TK,N} = \frac{B_{d,TKN,ZB}}{B_{R,TKN}} \quad [m^3] \quad (4.7)$$

Where

$B_{d,TKN,ZB}$	=	TKN load inflow	[kg/d]
$B_{R,TKN}$	=	Volumetric TKN loading	[kg/(m³*d)]

The total volume for trickling filters with nitrification is the sum of $V_{TK,C}$ and $V_{TK,N}$.

$$V_{TK} = V_{TK,C} + V_{TK,N} \quad [m^3] \quad (4.8)$$

The BOD_5 concentration of the influent should be less than $C_{BOD,VB,RF}$ 150 mg/l. The concentration can be adjusted with the recirculation of filter effluent. Equation 4.9 describes the relation between recirculation and inflow concentration.

$$RV_t \geq \left(\frac{C_{BOD,ZB}}{C_{BOD,RF}}\right) - 1 \quad (4.9)$$

The recirculation volume influences the total daily inflow volume. The total maximum dry weather inflow volume is calculated with equation 4.10.

$$Q_{TK} = Q_t * (1 + RV_t) \qquad [m^3/d] \ (4.10)$$

The filter surface load is determined with equation 4.11.

$$q_{A,TK} = \frac{Q_T*(1+RV_t)}{A_{TK}} \qquad [m/h] \ \ (4.11)$$

Where

Q_T = maximum dry weather inflow [m^3/h]

RV_t = recirculation (Q_{RF} / Q_t)

A_{TK} = filter surface area [m^2]

Surface load should be minimum 0.4 m/h.

The required area is a result of volume and surface area and calculated as follows

$$A_{TK} = \frac{V_{TK}}{h_{TK}} \qquad [m] \qquad (4.12)$$

Trickling filters with a height of 4 m have shown good results (DWA, 2001).

An even distribution of the water over the filter is important for its operation. Therefore, spray nozzles must provide enough power to spread the water consistently over the filter. The spray power is calculated as follows.

$$S_K = \frac{q_{A,TK}*1000}{a*n} \qquad [mm/arm] \qquad (4.13)$$

Where

$q_{A,TK}$ = Surface load trickling filter [m/h]

a = number of arms

n = rotations per hour [1/h]

4.4.1.4. Free-floating treatment wetlands

Free-floating treatment wetlands (FTW) are a combination of open water pond structures and floating island structures (FIS) that contain emergent plants. This structure has significant advantages to other wetlands, as it can handle and buffer bigger flow variations than wetlands containing media beds. According to Dodkins and Mendzil (2014), floating islands' root structure provides additional surface for microorganisms and supports BOD and nitrogen removal (Dodkins & Mendzil, 2014).

FTWs can be added onto existing ponds without elaborately changes of the existing structure. Even building costs are lower compared to other wetlands (Headley & Tanner, 2006; Dodkins

& Mendzil, 2014). While conventional wetlands often tend to clogging when the inflow nutrients load increases, FTWs can potentially handle high loads, as they are known for their higher nutrient removal capacity. The exposed roots support the sedimentation and filtration process, which reduces the turbidity (Dodkins & Mendzil, 2014).

Operation

For its treatment efficiency, an appropriate wetland design is necessary and will have a huge impact on the treatment pond's environment. Dodkins and Mendzil point out, that flow volume and flow variation have a significant effect on the hydraulic design of a wetland (Dodkins & Mendzil, 2014).

Pollutant concentration of the inflow can have an impact on the treatment performance (Chen et al., 2016) and required outflow characteristics should be considered in the wetland design (Dodkins & Mendzil, 2014). Van de Mooretel et al., 2010 and Pappalardo et al., 2017 mention, that pollutant removal efficiency also depends on climatic conditions and the right choice of wetland plant species (Van de Mooretel et al., 2010; Pappalardo et al., 2017).

According to Dodkins and Menzel, FTW are known to have a greater nutrient removal potential when the water's nutrient concentration is higher. Phosphorus removal is mainly based on adsorption and sedimentation where nitrogen removal occurs due to microbiological activity. FTW can be designed to provide either aerobic or anaerobic treatment conditions. It is important to specify which treatment goals the operator wants to achieve.

A coverage of more than 50 % up to 100 % of the pond surface creates anaerobic conditions and supports denitrification. A lower coverage than 50 % provides aerobic conditions and supports ammonia removal due nitrification. A coverage of 20 % is recommended to avoid anoxia occurring. FTWs are known to decrease the dissolved oxygen level and the redox potential in the pond. Additional aeration may be needed for nitrification, to provide sufficient oxygen transfer, which is required to oxidize Ammonia to Nitrate. In case of additional and sufficient aeration, the pond's plant coverage can be up to 100 %. In some cases, extra carbonate in form of calcium carbonate $CaCO_3$ can be added to improve the nitrification process. To support nutrient distribution and nutrient supply for microbiological processes, mixing the pond mechanically can aid (Dodkins & Mendzil, 2014).

The pH-level has a major impact on the functioning of a wetland. Floating wetlands are known to reduce the pH-level, due to the release of humid acids through the roots of wetland plants. This pH-decreasing effect leads to increasing denitrification. If the wetland should mainly provide denitrification, additional carbon can be added to achieve higher denitrification results. A combination of aerobic and anaerobic milieu can achieve better overall nitrogen removal rates.

The treatment performance of FTWs can also variate with the hydraulic retention time; longer retention times can affect a higher treatment efficiency. The nutrient uptake of wetland plants is not from significance for the treatment efficiency and provides mainly a habitat for microorganisms, which settle on their roots. To avoid a deposition of senescent material and additional nutrient intake into the water, wetland plants should be harvested within their seasonal life cycle. The method of harvesting depends on the wetland design. Plants can either be harvested on site or island segments can fully be replaced (Dodkins & Mendzil, 2014).

The treatment efficiency variates with the seasons, due to changing microbiological activity based on temperature variations. So does the DO level, as oxygen has a temperature depended solubility in water, and is affected by microbiological activity.

Due to Dodkins and Mendzil, FTWs have the ability to reduce temperature variations within a treatment pond, especially during summer when sufficient plant coverage exists (Dodkins & Mendzil, 2014).

4.4.1.5. Hybrid Wetlands

Hybrid wetlands are commonly known as a combination of different wetland structures. An advantage of hybrid wetlands is that combining different wetlands can complement their single disadvantages and achieve higher removal potential. The most common hybrid wetland is a combination of a horizontal flow and a vertical flow wetland (Kadlec & Wallace, 2009; Barco & Borin, 2017). Horizontal flow wetlands provide reasonable results for treating municipal wastewater but show low potential to remove Ammonia. Therefore, it is beneficial to combine it with a VF wetland, which is known for a great Ammonia reduction potential. Both wetlands can be operated either as series or parallel configurations.

Surface floating wetlands are modern types of wetlands and can be considered as hybrid wetlands as well. As research from Barco and Borin, 2017 shows, a combination of horizontal flow and surface flow wetlands has shown positive results (Barco & Borin, 2017).

4.4.2. Filter Media

The filter media is particularly important for HSSF and VF wetlands, as the biological and physical treatment occurs inside the main filter layer (Kadlec & Wallace, 2009; Geller & Höner, 2003). For effective treatment the contact time between wastewater and media is critical. The media should provide the capability for a continuous flow through the filter and a sufficient hydraulic retention time. Therefore, the permeability of the filter media is significant for a consistent flow. If a fine material with a low permeability is used, the risk of a blocked filter or an overflow is potentially higher. Using rough material can minimize the risk of blockages but can lead to a shorter retention time in the filter bed. In addition, the total surface for microorganisms to grow on is less in that case, which can lower the treatment potential of the wetland. The choice of the material should consider a sufficient permeability on one and enough surface area on the other side to provide a well performing wetland (Geller & Höner, 2003).

The k_f value in m/s is used to describe the permeability, and is variable depending on the grain grade. DWA and Geller and Höner recommend a main filter bed permeability in a range of 10^{-3} to 10^{-4} m/s (DWA, 2006; Geller & Höner, 2003).

According to ATV Worksheet A 262, the k_f – value can be calculated as follows (DWA, 2006).

$$k_f = \frac{(d_{10})^2}{100} \qquad (4.14)$$

where

d_{10} = grain diameter at 10% passing

To analyse the grain, filter material is commonly sieved. The d_{10} value describes the particle diameter mass distribution, where exactly 10 % of the grain is smaller than the d_{10} value grade (Geller & Höner, 2003). DWA recommends a filter material grain grade in a range of ≥ 0.2 mm to ≤0.4 mm (DWA, 2006). The amount of silt and clay inside the filter media considers particles smaller than 0.063 mm (Geller & Höner, 2003) and should be less than 2 % in total (DWA, 2006). It is mentioned, that a contribution of different sized particles also has an impact on the permeability. Therefore, the relation between d_{10} and d_{60} describes the uniformity coefficient C_u (Geller & Höner, 2003) and can be calculated according to ATV Worksheet A 262 (DWA, 2006).

$$C_U = \frac{d_{60}}{d_{10}} \qquad (4.15)$$

where

d_{60} = grain diameter at 60% passing

d_{10} = grain diameter at 10% passing

The filter media should have a uniformity coefficient less than 5 (DWA, 2006; Geller & Höner, 2003). When grain size is almost even for all grains, its permeability will be higher. An uneven distribution of the grain size can cause accumulation of smaller particles on the bottom of the filter bed, which can cause permeability reduction of the filter. The filter material should provide sufficient sorption potential for Ammonia and Phosphate. Longer sorption will supply better results for Ammonia and Phosphate reduction. Material with a higher cation exchange capacity will show greater Ammonia removal. However, expanded clay is known to have a good capability to absorb Ammonia, which supports nitrogen removal.

Furthermore, the filter material should provide a buffer capability to buffer the filter bed's pH-level. Low pH levels can lead to a decreasing microbiological activity. Material that contains great amounts of magnesium carbonate is known to supply great buffer capacity, whereas material containing calcium carbonate should not be used as calcium carbonate can be degraded with low pH, and blog the filter (Geller & Höner, 2003).

Sand or gravel with a permeability of k_f= 10^{-3} to 10^{-4} m/s is commonly used for vertical and horizontal flow wetlands (DWA, 2006). The New Zealand National Institute for Water and Atmospheric Research recommends using a gravel media with a particle size of 10 to 20 mm with a porosity of 40 % for horizontal subsurface flow wetlands (Tanner et al, 2011).

4.4.3. Wetland plants

Wetland plants are particularly important for an efficiently working constructed wetland. They offer a physical structure for microbial biofilms to grow on. Wetland plants can supply denitrification with the addition of organic material. According to Tanner et al, density and size of plants can have an impact on water temperature, as the plant proliferation shade the water surface (Tanner et al, 2010).

A wide range of wetland plants have been used in constructed wetlands. The choice of species decides upon the success of natural water treatment. Species must be suitable for the local climatic conditions and must be adapted to the actual wetland design. Scientists recommend selecting locally occurring native plants (Geller & Höner, 2003; Tanner et al, 2006).

In the table below, native plants and their applications for wetlands are listed.

Plant species	Description and characteristics	Application
Baumea articulate  http://www.nznativeplants.co.nz/shop/Grasses+reeds++rushes/Baumea+articulata.html	• Also known as "jointed twig-rush" and grows from Northland to south of Levin • Evergreen plant, 1.8-2.0 m tall and 0-0.4m deep roots • Suitable for surface and subsurface-flow wetlands • Takes two growing seasons to develop (slowly compared to other wetland plants) • Usually planted in combination with *S. tabernaemontani* (Tanner et. al, 2006)	Surface & subsurface wetlands
Carex secta  http://www.terrain.net.nz/friends-of-te-henui-group/plants-grasses-sedges-rushes-nz-natives/carex-secta-pukio.html	• Native plant growing all over NZ and known as purei, makura or niggerhead • Evergreen plant, which is 1-1.5m tall with roots growing 0-0.2m deep • Suitable to grow on gravel-bed constructed wetlands and margins, shallow zones and embankments of surface flow wetlands • Young plant shouldn't be established in water deeper than 100mm, mature plant can grow in deeper water • Common plant used for wetland and stream margins (Tanner et. al, 2006)	Surface & Subsurface flow wetlands
Eleocharis sphacelata  http://www.nzpcn.org.nz/flora_details.aspx?ID=2123	• Known as Kta, tall spike-rush or spike-sedge, common on North Island but suitable for whole New Zealand • 0.8-1.3m tall with bright green leafless shoots, 0.6m deep roots (one of the deepest growing wetland plants) • Establishes quickly and has a traditional importance for Maori (Tanner et. al, 2006)	Surface flow wetlands
Schoenoplectus tabernaemontani  http://www.nzpcn.org.nz/flora_details.aspx?ID=2252	• Called kapungawha, soft stem bulrush or lake clubrush and grows from southern Northland to the Westland and in the Canterbury region • 0.6-1.8m tall with blue-green leafless hollow shoots • Seasonal plant, grows well in warmer coastal zones (established fast during spring/summer and dies back during winter) • Most common species for wetlands • Should be combined with evergreen plants that survive winter (Tanner et. al, 2006)	Surface & Subsurface flow wetlands
Typha orientalis 	• Commonly known as raupo or bulrush, native plant all over NZ, can be found in fertile lowland swamps all over NZ • Seasonal plant, grows during spring and summer with a height up to 1.5 - 3m • Tends to produce large accumulations of standing and decomposing litter • Traditionally used by Maori for thatching ant eating (Tanner et. al, 2006)	Surface flow wetlands

http://ketenewplymouth.peoplesnetworknz.info/image_files/0000/0007/7874/Cylindrical_seed_heads_of_Typha_orientalis._Bulrush__raupo-5.JPG		

Table 4-1 Wetland plant species

After selecting wetland plants, establishment and planting is the next important step. Therefore, several things must be considered to ensure successful establishment of the plants. The right time of the year is essential for planting, as seedlings and young plants are particularly sensitive to environmental conditions. Most wetland plants do not grow during winter. Best suitable conditions for planting are during spring and/or early summer. Good establishment results can be achieved by using nursery stock grown plants from seeds (Tanner et. al, 2010). It is also possible to harvest and transfer natural growing plants into the wetland system, which usually requires a permission (Tanner et al, 2006). The plants should have well developed roots and rhizomes before transferring into a wetland (Tanner et. al, 2010).

4.4.4. Pre-treatment requirements

Before water is introduced into a constructed wetland, it should be pre-treated to remove a majority of suspended solids. A high number of suspended solids could cause blockage of the filter material. Pollutants can be degraded during the pre-treatment process as well. This depends mainly on the hydraulic retention time during pre-treatment (Geller & Höner, 2003). According to the DWA, 2006, the total filterable solids introduced by the influent into a constructed wetland including a filter bed should be less than 100 mg/l (Geller & Höner, 2003; DWA, 2006). To pre-treat influent, a preferred procedure for more than 100 connected population equivalents can be used either an Imhoff tank or a sedimentation pond (Geller & Höner, 2003).

According to DWA-ATV worksheet A 201, sedimentation ponds are designed with a capacity of minimum 0.5 m^3 per capita. An optimum would be 3-4 m^3 per capita. The hydraulic retention time should be minimum one day during dry weather flow (DWA, 2005).

An Imhoff tank is designed as minimum 120 l per capita, which is separated in 30 l for sedimentation cell, 30 l for floatation cell and 60 l for digestion cell (Geller & Höner, 2003).

4.4.5. Potential of wetlands

Constructed wetlands are known to have a great pollutant removal potential (Kadlec & Wallace, 2009; Geller & Höner, 2003; Headley & Tanner, 2006). Different kinds of wetlands have been researched in the last years. Research about using wetlands for natural water treatment mainly includes mesocosm studies (Pavlineri et al., 2017). Studies found in the literature are difficult to compare and a type of wetland cannot be generalized as these systems are complex and influenced by various environmental factors (Kadlec & Wallace, 2009). However, many successful operating wetlands can be found in literature.

Good removal results have been found for hybrid wetlands. The combination of HSSF and VF wetlands was successful in many cases, as studies from Ayaz et al., (2015) with decreasing results of TN: 19-66%, BOD: 70-91%, COD: 79-92 and Nyakang'o and Van Bruggen (1999) with resuts of 90% TKN, 92% NH_4-N, 88% PO_4-P and 96% COD show. Another study from Ye and Li (2009) shows results of 82 % TN, 84.6 % COD, 64.2 % TP.

The setup of HSSF and VF Wetland combined with an Imhoff tank as pre-treatment and a FWS wetland, achieved remarkable results of TN 94.5%, NH_4-N: 97.6%, TP: 47.5%, PO_4-P: 15.6% and COD 89.4% reduction (Avila et al.,2015). Wetland setups pre-treated with an Imhoff tank have shown great results through to the literature (Avila et al., 2013; Avila et al., 2015). Free-floating wetlands have shown good performance reduction results of TN: 72.7%, NH_4-N 75.8%, and COD 94.6% (Xin et al., 2012).

Many cases of great pathogen reduction potential have been reported, such as pathogen removal efficiencies ranging up to 99.9 % (Kadlec & Wallace, 2009; Boutilier et al, 2009). The microbiological removal performance of wetlands is particularly high during high influent concentrations of 10^5-10^6 cfu/100ml. For inflow concentrations below 10^3 cfu/100ml no significant removal occurred (Hagendorf et al, 2004; Kadlec & Wallace, 2009).

In addition to water treatment achievements, wetlands as natural water treatment systems that enable water to get in touch with the earth could satisfy local cultural values, which needs to be considered while developing an alternative wastewater treatment option for Wairoa.

5. Design parameters and considerations

5.1. Location

5.1.1. Hydraulic loadings

Flow data of daily volumes and flowrates are measured, monitored and logged by Wairoa District Council, using a Supervisory Control And Data Acquisition (SCADA) software, logged on a main SCADA server. All used data is downloaded from the main server and edited with Microsoft EXCEL.

For this thesis, all data has been assessed according to worksheet ATV-DVWK-A 198. Median, maximum, minimum and percentile values have been determined for dry weather flow (DWF), Wet Weather Flow (WWF) and combined weather flow. Weather data for the determination of WWF have been taken from a Hawkes Bay Regional Council database.

For calculating the required flow parameters, data from 2016 to 2018 has been analysed and evaluated.

Daily flow and flowrate

The daily inflow volume and flow rates for the period of 1.1.2016 to 1.6.2018 are summarised in table 5-1. Daily inflow data is separated in Dry Weather Flow (DWF), Wet Weather Flow (WWF) and All Weather Flow (AWF). All Weather Flow is a term for total data, combining DWF and WWF. Inflow rate data is only summarised according to the AWF. Values for the median, 10th, 85th and 99th percentile have been used to characterise the flows. The inflow volume into the wastewater treatment plant equals the export volume of Fitzroy pump station, which is measured by a flowmeter located at the pump station outlet.

Flow Type	**Daily Inflow 2016 - 2018**		**Inflow rates 2016 - 2018**
	m^3/d		l/s
AWF	median	2,284	35
	10th percentile	1,527	14
	85th percentile	4,117	70
	99th percentile	6,342	90
DWF	median	2,176	-
	10th percentile	1,546	-
	85th percentile	3,801	-
	99th percentile	6,247	-
WWF	median	2,634	-
	10th percentile	1,252	-
	85th percentile	5,032	-
	99th percentile	6,397	-

Table 5-1 Daily mean inflows based on Fitzroy pump volumes per day

Flow variability/distribution

Table 5-2 highlights the distribution of the flowrate into the Wairoa Wastewater Treatment Plant for the years 2016, 2017 and 2018. Usually, the flowrate is in a range between 30 l/s and 60 l/s. During heavy rainfall it can increase up to 90 l/s. In certain events the flowrate is greater than 90l/s. During dry periods the inflow rate can be less than 30 l/s. The relation between rain and flow variation is shown in Appendix B.

Daily Mean Flows (L/s)	Daily Flow Distribution
<30	33%
30-60	51%
60-90	15%
>90	1%

Table 5-2 Flow distribution daily inflow from Fitz Roy for 2016 -2018

5.1.2. Water quality monitoring results

Wairoa District Council's wastewater monitoring program includes a 24-hour time proportional sample of the wastewater effluent. From 2008 until 2017 WDC also measured the influent quality. Results, that are listed in table 5-3, are the only available data about chemical and biological conditions of influent after screening and effluent.

Parameter	Unit	Inflow*1			Outflow*2		
		Median	85th percentile	95th percentile	Median	85th percentile	95th percentile
pH*3	-	7.5	-	-	7.6	-	-
Dissolved oxygen	g/m^3	2.95	4.5	5.1	6.2	12.0	16.5
Turbidity	NTU	68.0	109,8	199.8	32.0	66.5	94
Total Nitrogen	g/m^3	23.0	38	43.8	-	-	-
Ammonia	g/m^3 as N	16.3	27.2	34	17.5	26.0	31.3
Nitrate + Nitrite	g/m^3	0.007	0.66	2.54	-	-	-
Total Kjeldahl Nitrogen	g/m^3 as N	23.0	38	43.8	-	-	-
Total Phosphorus	g/m^3 as P	3.4	5.2	6.66	-	-	-
$CBOD_5$	g/m^3 as O	78.5	150.95	263.5	23	44.6	69.9
COD	g/m^3 as O	240	410	668	110	220	323
e. coli	Cfu/100ml				1,800	78,750	240,000

Table 5-3 Wastewater chemical monitoring data

*1 Data available 4.2008 – 12.2017, *2 Data available 1.2000 – 5.2018, *3 Min pH 6.3, Max pH 9.2

5.1.3. Current system performance assessment

To access any further changes, it is necessary to validate the status of the existing system. Therefore, hydraulic retention time, nutrient loading and removal rate has been accessed. Calculations have been made according to Worksheet DWA A 201. Flow data has been accessed according to the DWA ATV-DVWK-A 198 for the standardization and derivation of rated values for wastewater treatment systems. Sludge accumulation and volumes have been measured by Parklink in 2016 (Appendix E). Wairoa District Council desludged the oxidation pond in April 2018 and removed approximately 518 m^3 of sludge (Appendix F).

Facultative Pond

- Approximate maximum total capacity of the pond 5,350 m^3
- Approximate wet sludge volume 2,750 m^3
- Approximately removed sludge volume 517 m^3
- Available Volume without sludge accumulation 3,117 m^3
- Percentage of pond occupied by sludge 58.9%

Maturation pond

- Approximate maximum total capacity of the pond 24,130 m^3
- Approximate wet sludge volume 5,863 m^3
- Available Volume without sludge accumulation 18,267 m^3
- Percentage of pond occupied by sludge 32.1%

85th percentile of All Weather Flow $Q_{awf,85th}$= 4,117 m^3 /d

Hydraulic retention time

DWA and NZWWA set hydraulic retention times for oxidation ponds, such as five days for a primary oxidation pond and one to two days for a maturation pond during dry weather flows (DWA, 2005; NZWWA, 2005).

According to the DWA (2016) formulary for “environmental occupations - wastewater technology”, the hydraulic retention time is calculated with equation 4.2.

The calculated hydraulic retention time for the oxidation pond is:

$$t_R = \frac{3{,}117 m^3}{4{,}117 \frac{d}{m^3}} = 18 \text{ h, } 10 \text{ min.}$$

The calculated hydraulic retention time for the maturation pond is:

$$t_R = \frac{18{,}267 m^3}{4{,}117 \frac{d}{m^3}} = 4 \text{ days, } 10 \text{ h, } 29 \text{ min.}$$

The hydraulic retention time of the oxidation pond is with 18 h and 10 min lower than the recommended five days for oxidation ponds. The maturation pond has a significant higher retention time than recommended (DWA, 2005; NZWWA, 2005).

Volumetric loading

A recommendation from DWA-A 201 says, the volumetric loading for facultative ponds should be $B_{d,BOD5} \leq 25$ g/(m^3*d). The BOD_5 (Biochemical Oxygen Demand over 5 days) of Wairoa's wastewater treatment has been monitored once a month, from 2008 until December 2017. The median for this period is 78.5 g/m^3, which is lower than the expected inflow load for raw sewer of 200 g/m^3 (Metcalf & Eddy, 2014). A low concentration of BOD_5 could be caused by high inflow and infiltration into the wastewater reticulation system.

Daily BOD_5 load according to the monthly influent monitoring would be calculated by

$$B_{d,BOD5} = c_{BOD5} * Q_{dw} \left[\frac{kg}{d}\right] \quad (5.2)$$

Where

$$c_{BOD5} = BOD_5 \; concentration \; influent$$

$$Q_{dw} = Dry \; weather \; flow$$

$B_{d,BOD5}$ calculated based on the monitoring data $\quad 78.5 \frac{g}{m^3} * 2{,}176 \frac{m^3}{d} = \mathbf{170.82} \frac{\boldsymbol{kg}}{\boldsymbol{d}}$

$B_{d,BOD5}$ calculated based on the BOD_5 loading per capita $\quad 60 \frac{g}{E*d} * 4{,}250\text{E} = \mathbf{255} \frac{\boldsymbol{kg}}{\boldsymbol{d}}$ (DWA, 2016).

According to the DWA formula, the volumetric loading can be calculated by

$$\boldsymbol{B_{R,BOD5}} = \frac{\boldsymbol{B_{d,BOD5}}}{\boldsymbol{V_R}} \left[\frac{\boldsymbol{kg}}{\boldsymbol{m^3*d}}\right] \quad (5.3)$$

where

$B_{d,BOD5}$	=	Daily BOD_5 load
V_R	=	Reactor volume

Based on monitoring data, the volumetric loading for the oxidation pond is

$$\boldsymbol{B_{R,BOD5}} = \frac{\mathbf{170.82}\,\frac{g}{d}}{\mathbf{3{,}117}\,\boldsymbol{m^3}} = \mathbf{54.8}\ \frac{\boldsymbol{g}}{\boldsymbol{m^3*d}}$$

Based on the theoretical value of 60 $\frac{g}{E*d}$ according to the DWA, the volumetric loading for the oxidation pond is

$$\boldsymbol{B_{R,BOD5}} = \frac{\mathbf{255}\,\frac{\boldsymbol{kg}}{\boldsymbol{d}}}{\mathbf{3{,}117}\,\boldsymbol{m^3}} = \mathbf{81.81}\,\frac{\boldsymbol{g}}{\boldsymbol{m^3*d}}$$

The median of inflow BOD_5-load of 78 g/m^3 is lower than expected, which can be a result of high inflow and infiltration of storm water into the sewer system (NZWWA, 2005).

Results for volumetric loading based on monitoring results and theoretical calculated $B_{R,BOD5}$, are higher than the recommended value of $B_{R,BOD5} \leq 25$ g/(m^3*d) (DWA,2005).

Nutrient removal performance

As the hydraulic retention time is not considered during sampling, no sufficient performance assessment can be made based on sampling data from table 5-3. However, it's the only available data, which gives a rough indication of how the system performs.

Data of discharging has been compared with typical effluent results for primary oxidation ponds of New Zealand (Appendix B). The comparison has shown, that only effluent Ammonia values are slightly higher than typical values, while overall, Wairoa's treatment performance is similar to the expected performance of various oxidation ponds in New Zealand.
A comparison between Ammonia influent and effluent data (Appendix B) shows no significant reduction. The median for Ammonia in the effluent is only slightly less than the influent one.

Reasons for a low removal rate can be a low concentration of the influent or a short hydraulic retention time in the oxidation pond.

5.1.4. Options and Availabilities

In New Zealand, constructed wetlands are often added to pond treatment systems for effluent polishing. They are providing advanced secondary or tertiary treatment and are mainly applied to address cultural issues and fulfil spiritual maori values (NZWWA, 2005). In the literature, design assumptions are often made for smaller scale wetlands as primary or secondary treatment (Kadlec & Wallace, 2009; Geller & Höner, 2003).

However, when developing options for Wairoa, some parameters are unknown, such as future discharge standards, required for the prospective Discharge Consent. Based on the assessment of the current system potential, improvements should be considered for further upgrade options. A significant effort should be made to address cultural values. The literature research about Maori values has shown that the interpretation of spiritual values offers a wide range of interpretation and is a complex topic. The most noticable feature of the water cycle is the relation of the sky father (Ranginui) and the earth mother (Papatuanuku) (Morgan, 2006; Ihaka et al, 2000; Ministry for the Environment, 2003). Spiritually damaged water must return to the earth mother to regain new life (Morgan, 2006; Douglas, 1984). Therefore, it can be concluded that any potential design should provide contact between water and earth. This feature points out that some kind of filter bed should be implemented in each design option to fulfil cultural requirements.

Each technical concept design should include considerations about pre-treatment requirements, to avoid negative impacts of solids on the system. Solids in particular can cause problems in wetlands, and decrease their performance (Kadlec & Wallace, 2009; Geller& Höner, 2003). Considerations could include using already existing infrastructure or building a new structure to remove solids from the influent, such as an Imhoff tank.
The existing pond structure does not include mechanically desludging. Resulting, high sludge accumulations has been noticed in the past (Appendix E). An upgraded solid removal could be beneficial for the whole system and support future sludge management.

As most of New Zealand's waste water treatment systems, Wairoa's system suffers from high inflow and infiltration (I&I) of stormwater (NZWWA, 2005; LEI, 2017). In chapter 5.1.1 it is noted that the inflow into the treatment plant varies along with weather conditions. During heavy rain events the daily inflow volume is approximately about three to four times higher than the normal daily inflow volume. High flow variations are a result of high I&I into the reticulation system (LEI, 2017) and can hydraulically overload the system (Kadlec & Wallace,

2009). To ensure a consistent wetland operation, flows may have to be buffered to balance the flowrate. The required buffer depends on the hydraulic design of each system. Hydraulic design will be created according to flow characteristics mentioned in chapter 5.1.1. Based on the assessment of the inflow data, a design for max 60 l/s would match 84 % of all flows, with 16 % flows, that needs to be buffered or bypassed. Wairoa District Council is currently working towards improvements to decrease I&I into the Wastewater system (LEI, 2017) which will reduce peak flows. It is expected that flows above 60 l/s will decrease during these improvements. To meet the hydraulic design criteria, the system must be able to operate at a maximum flowrate of 60 l/s. This could implement a "one system"-solution based on 60 l/s maximum flow or a "two system"-solution, each suitable for 30 l/s. Furthermore, modulation of smaller systems with lower hydraulic requirements could be an option. The choice of operation will also affect the wetland design. A design with several individual systems would allow an alternating operation and provide the capability to maintain one system while another one is operating. For "multi system"-solution must be proofed if there is sufficient water supply to maintain potential wetland plants with water during summer times when inflow volumes into the Wairoa treatment plant are usually lower (Appendix B).
Commonly, when designing wastewater systems, the 85th percentile is used (DWA, 2003). The 85th percentile of the inflow into the wastewater treatment plant is 4,116 m^3/d. In Appendix B, the water consumption of Wairoa town is assessed. The 85th percentile of Wairoa's water consumption equals 1,455 m^3/d (Appendix B). The 85th percentile of the daily pumped wastewater is about the factor of 2.8 higher than the consumed potable water. The volume of storm water entering the reticulation system is expected to decrease in the future. Therefore, some design assumptions are based on the all-weather median flow of 2,284 m^3/d.

Which type of wetland is suitable mainly depends on the intended treatment goals. The assessment of the current system has shown a deficit of nitrification. To improve the nitrification rate and remove Ammonia more efficient, the design could include a Vertical Flow Wetland which is known for good nitrification potential (Kadlec & Wallace, 2009; Geller & Höner, 2003). VF wetlands require 20 % less surface than HSSF wetlands, which is important as lack of space is a restricting factor. A combination of VF wetland and HSSF could provide additional Nitrogen removal through HSSFs denitrification potential. It should be examined if a combination of VF and HSSF has a bigger potential to address cultural values, as it could potentially imitate the natural water cycle more adequate than a single system and creates longer contact times between earth and water. A floating wetland structure could be complemented with any wetland system or be an option on its own. Disadvantageously, an exclusively application of a floating wetland structure could be a short contact between water and earth to simulate the natural water cycle.

Councils from other districts, such as Gisborne and Napier are utilising trickling filters to upgrade their treatment plants (Napier City Council, 2018; Gisborne District Council, 2016). In general, trickling filters are not considered to be natural treatment plants but potentially combine cultural values and possible design aspects. Using rocks as filter media provides contact between earth and water. Interpond rock filters can be an option to upgrade a treatment plant without much construction works.

5.1.5. Discharge environment

The discharge environment changes, due to tidal influence. The water depth varies between 1 and 2 m during periods of high tide. During low tide the discharge port is usually dry or at the water level as silt heavily accumulates in the surrounding area (EAM, 2012). According to New Zealand's estuary classification system the Wairoa River Estuary is classified as an "F estuary", which is characterised by a spit or shingle bar enclosing a large primary basin with numerous of arms lead off (Hume et al. 2007). The connecting river mouth of the Wairoa River estuary and the Pacific Ocean is influenced by changing currents and moves over the times. Silt accumulations at the sides of the lagoon tend to be muddier and the centre of the lagoon sandier. Water movement in the lagoon is mainly influenced by tides. River water entering the lagoon over a tidal cycle is typically small compared to the total volume of the basin (EAM, 2012). An effluent dilution study from 2007 has considered the "worst-case scenario" of a blocked river mouth and showed that the mixture of effluent and river water is only influenced by wind dilution. A dilution of 5:1 within 150 m around the discharge port has been expected and could cause significant human health risks when using the lower estuary. Under normal flow conditions, the dilution at the discharge port was about 5:1, 125 m around the discharge was diluted 50:1 and 350 m downstream 250:1 (Barter, 2007).

5.2. Design assumptions

5.2.1. Land area needed

Every wetland requires a different amount of space, depending on type and hydraulic design. First assumptions about required surface area for each wetland type can be made according to the connected residents (DWA, 2006; Geller& Höner, 2003). As mentioned in Worksheet DWA-A 262, the required area needed for a wetland can be calculated by the number of

connected residents, regarding horizontal subsurface flow wetlands with 5 m^2 per capita and vertical flow wetlands with 20% less than HSSF wetlands (4 m^2 per capita).
Calculated with these reference numbers and a population of 4250 residences, the required area would be **21,250** m^2 for an HSSF and **17,000** m^2 for a VF wetland.

Daily surface load according to the total daily inflow volume and specific filter surface are further parameters to identify the needed area. Different surface loadings have been calculated to determine the relation between area surface and daily total inflow volume (Appendix D).

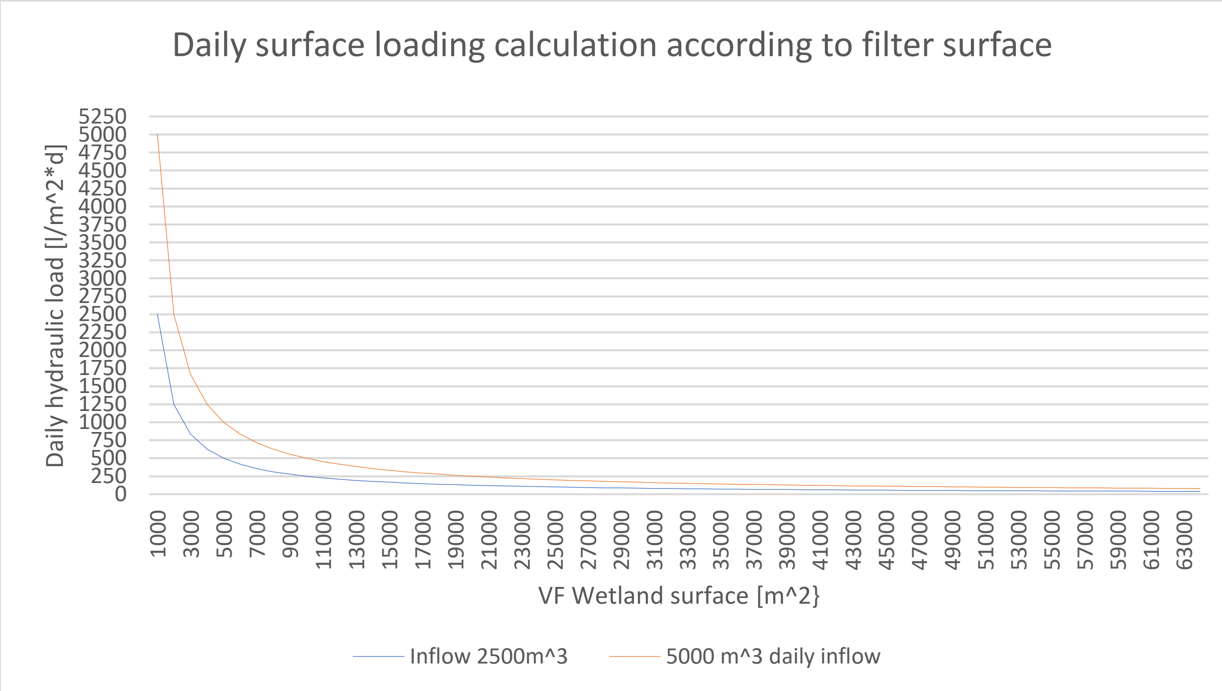

Table 5-4 Daily surface loading according to filter surface

Table 5-4 shows two examples of the relationship between inflow, filter surface and surface load. The two graphs represent the total daily inflow volume of 2,500 m^3 (blue graph) and 5,000 m^3 (orange graph) and show the relation between surface load and wetland surface. The calculation points out that the surface load for filter with an approx. surface < 3,000 m^2 have high surface loadings >> 1,000 l/m^2*d. The surface load decreases rapidly with an increasing filter surface in a range of 1,000 m^2 to 10,000 m^2.
A surface load for a VF wetland with a surface of 1,000 m^2 and a daily inflow rate of 2,000 m^3 would have a surface load of 2,500 l/m^2*d. A wetland of 10,000 m^2 would only have a surface load of 250 l/m^2*d. A VF wetland with a maximum inflow rate of 60 l/s (equal to 5,148 m^3 /d) needs a minimum surface of 63,000 m^2 to observe the maximum surface load of 80 l/m^2*d.

Recommended values for required surface per connected resident and the maximum surface load found in the literature relate to small scale wetlands. The calculated required land area based on DWA recommendations would exceed the available land area of Wairoa District Council about nine times. However, it must be kept in mind that wetlands in the literature are usually considered to be used as primary or secondary treatment systems (DWA, 2005).

As conventional designed wetlands require more area than available on Council's property, a natural water treatment design must be smaller in its footprint. Another option would be introducing a rock filter, which is commonly designed according to wetland guidelines in New Zealand (Crites et. al. 2014). Rock trickling filter require smaller surface areas than wetlands (Metcalf & Eddy, 2014; Crites et. al. 2014) and provide contact between water and "earth".

5.2.2. Water distribution system

Planning and designing a water distribution system depends on the type of wetland that is chosen for construction. Each system requires its own design. For each kind of distribution system, it must be ensured that the influent is distributed steadily over the filter bed to provide a constant surface loading.

Sprinkler for example, can be a suitable distribution system for vertical flow wetlands. They provide contact time between air and water before soaking into the wetland system, which can be symbolically seen as a relation between sky (Ranginui) and earth (Papatunaku).

As settable solids, oil and grease can plug the sprinklers, wastewater must be pre-treated sufficiently. The biggest particle in a sprinkler system should be maximum one third of the diameter of a sprinkler nozzle (Crites et. al. 2014).

5.2.3. Lining requirements

Lining requirements for wetlands can be found in Worksheet DWA 262. According to this, wetlands must include a liner unless the soil permeability is $k_f < 10^{-8}$ m/s. PE liner with a thickness of > 1.0 mm are most commonly used. Lining material must be resistant to UV-light and strong enough to resist roots. Using concreted or plastic ponds is another option to fulfil lining requirements (DWA, 2006).

As mentioned in 3.2.1, the soil permeability of Awamate silt loam is less than 4 mm/hour, equals $1.1*10^{-6}$ m/s, which means it is necessary to use a liner when building a wetland structure.

6. Development and assessment of wetland designs for Wairoa

In the previous chapter, conditions and requirements for a wetland design have been identified. According to guidelines for constructed wetlands, the required land area has been calculated. It was found, that the land area needed is significantly higher than available. The following chapter contains design options, which are designed side specific, considering limited construction land.

6.1. Option 1

To satisfy cultural requirements, the first option is aiming to simulate a natural water cycle.

The treatment design includes a vertical flow rock filter wetland at its first stage and a horizontal flow wetland as a secondary treatment process.

In the following, a scheme of the system (figure 6-1) and a detailed description will be presented.

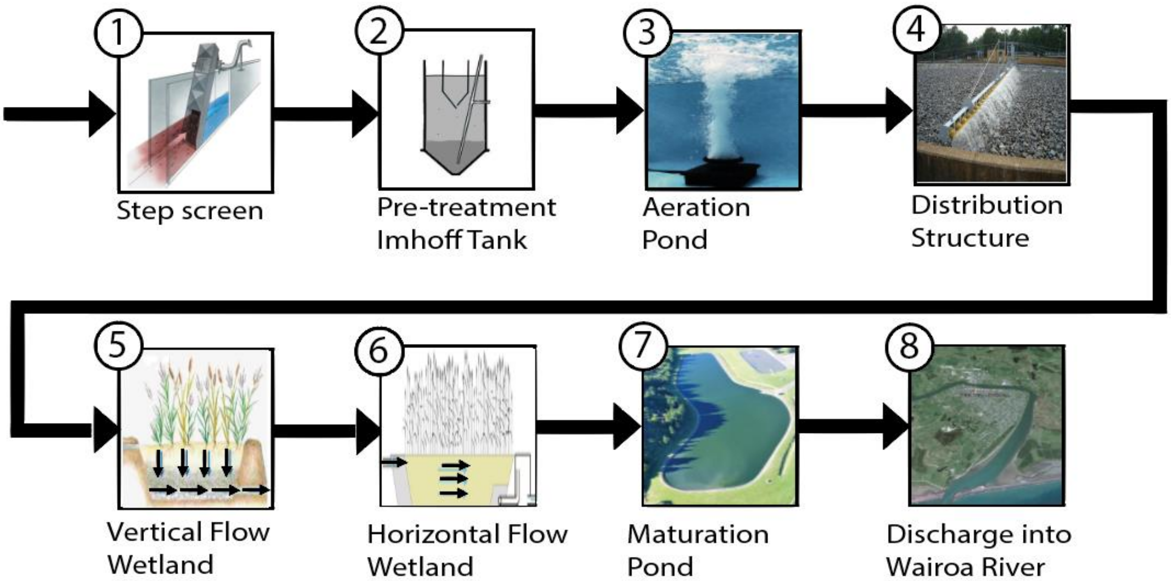

Figure 6-1 Flow scheme of Option 1 (own graphic)

Influent is pumped from the pump station to the treatment plant, where water enters a 5 mm step screen before it flows into an Imhoff tank (Step 1). All solids greater than 5 mm are screened. Sand, oil and solids smaller than 5 mm can pass the screen.

The Imhoff tank is applied as a pre-treatment pond (2). Floatable substances are separated from water, suspended solids settle down and can be discharged and stored for further treatment. Anaerobic digestion can occur in the sludge chamber.

The water then enters the aeration pond (3), where oxygen is added and nitrification occurs. In case of high inflows, the existing overflow weir provides an overflow into the maturation pond and bypasses the wetland.

In the next step, aerated water is pumped from the aeration pond uphill to the first natural treatment design. A rotator diffuser system (4) spreads the water over a circular vertical flow wetland (5). The diffuser rotates around its own centre and pumps water through a series of holes in its arms. Through water's power, the rotator can be operated. The distributed water travels in vertical direction, from the surface to the bottom of the filter. The first filter layer contains gravel, a support layer on the bottom avoids the outwash of smaller particles. Through gravity, the discharge flows from the bottom of the wetland, which has its lowest point at the centre, into the horizontal flow wetland (6). Water is distributed through a pipe reaching the whole width of the wetland and then travels through the filter bed in horizontal direction. From there it is discharged into the maturation pond (7) and subsequently into the Wairoa River (8).

Assessment of the system

As already mentioned in chapter 5.2.1, the area needed for a wetland designed after common design parameters requires approximately nine times more than available (required area according to DWA 63,000 m^2, available area approximately 7,000 m^2). Even though, the wetland design provides water and earth contact, which is from significance for spiritual cleaning, and suits the hydraulic requirements.

A design schematic is shown in figure 6-2. It can be seen as a hybrid wetland, as it combines a vertical and horizontal flow wetland. The concept's idea is to represent the natural water cycle, it includes "rain" (diffuser system, no. 4) which falls on and travels through the "earth" (wetlands, no. 5 and 6).

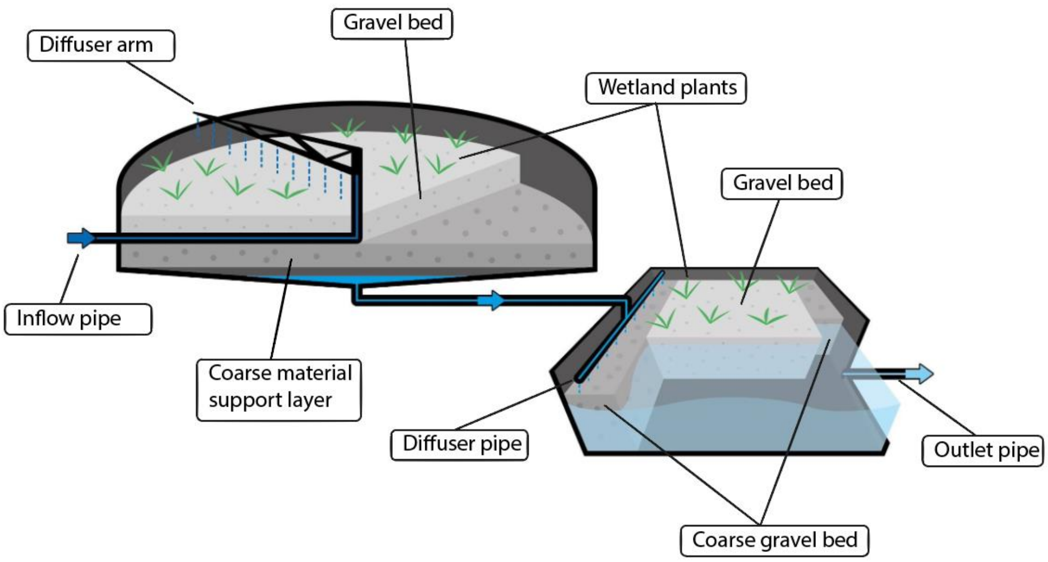

Figure 6-2 Wetland scheme of Option 1 (own representation)

The vertical flow wetland (Step 4 and 5) is the first step of natural water treatment. Water needs to be pumped uphill, from the aeration pond to the wetland. A two-arm rotating diffuser system irrigates the water evenly on the filter surface. The wetland is bedded in a circular pond, mainly as it is easy in operation and maintenance.
According to GIS mapping the biggest available diameter on Council's property is 70 m. An image of dimension and positioning is shown in figure 6-3.

Figure 6-3 Potential position of the VF wetland (GoogleMaps, own representation)

The wetland can be designed with an approximate surface of 3,848 m^2. A maximum flowrate of 60 l/s is expected, the maximum daily volume would be 5,184 m^3. If the flowrate exceeds 60 l/s, the wetland can be bypassed into the maturation pond.
The maximum daily surface load is calculated according to equation 4.1:

$$q_{A,d} = \frac{5184 \frac{m^3}{d}}{3848\, m^2} \qquad (6.1)$$

According to the calculation, the maximum surface load is 1.35 m/d equal to $1.57*10^{-5}$ m/s, at an inflow rate of 60 l/s. The commonly used material for wetlands has a permeability of $k_{fA} \geq 10^{-3}$ m/s, the material used for Wairoa's potential vertical wetland must be $k_{fA} \geq 1.56*10^{-5}$ m/s.

According to DWA-A 262 the required grain size is determined with equation 4.14:

$$d_{10} = \sqrt{1.56 * 10^{-5}\frac{m}{s} * 100}$$

Consequently, maximum 10 % of the material must be smaller than d_{10}= 1.25 mm. A summary of hydraulic conductivity is listed in in Appendix G.

Sand with a grain size of 0.06 - 2 mm is listed with a hydraulic conductivity of 1 - 5 m/d. Before using sand, the hydraulic conductivity should be examined in a trial. Alternatively, gravel can be used. According to recommendations of the National Institute of Water and Atmospheric Research, gravel of a grain size 2 - 64 mm has a hydraulic conductivity of $5*10^2$ m/d to $1*10^4$ m/d. Smaller grained gravel would meet the hydraulic requirement and is less sensitive to clogging than sand.

A support layer of 0.2 m minimum depth avoids outwash of filter material. Local material like pumice rock can be used, which is a kind of local volcano rock and a commonly used building material. A filter layer depth of 2 m and 0.5 m support layer are supposed. With a total height of 2 m and a surface of 3,848 m^2, the volume of the filter layer comes to 7,697 m^3.
The porosity of gravel (grade 2-64 mm) is with 0.25-0.35 slightly less than sand (grade 0.06-2 mm) with 0.30 – 0.40. As gravel has a better hydraulic conductivity and a similar porosity, it is the preferred material.
With a maximum porosity of 0.35, gravel would provide 2,694 m^3 void space. At its maximum inflow of 5,184 m^3/d, the wetland would theoretically provide a hydraulic retention time of 12 hours and 28 minutes. The hydraulic retention time for the median flow of 2,284 m^3/d would be one day, 4 hours and 18 minutes. To avoid uncontrolled runoff, the wetland should have a control outflow valve.

The vertical wetland's outflow structure has to provide a drainage ability greater than 60 l/s to drain the wetland.

The runoff then enters the horizontal flow wetland by a distributer pipe, which irrigates influent through several ports over the whole width of the wetland. With a maximum width of 100 m, it can be considered to divide the wetland into different chambers to provide even distribution. Width and location are shown in figure 6-4. An explanation of the dimensions can be found in Appendix J.

Figure 6-4 Potential location of the HSSF wetland (GoogleMaps, own representation)

By DWA-A 262, a length of 3 – 6 m is recommended for horizontal subsurface flow wetlands. The minimum required height for the filter layer is ≥ 50 cm. A design depth of 1 m and a height difference of inlet and outlet of 0.1 m is supposed. The total face area can be designed with 100 m^2. The required k_f value for the wetland is determined by equation 4.3 and equation 4.4 for the hydraulic gradient.

$$k_f = \frac{Q*l}{F*\Delta \mathrm{h}}$$

$$k_f = \frac{0.06\ \frac{m^3}{s} * 5m}{100\ m^2 * 0.1\ \mathrm{m}}$$

The calculated k_f value for the filter material is 0.03 m/s. According to DWA-A 262, the real k_f value should be by the power of ten higher than the calculated one. The filter material should have a permeability of 0.3 m/s equal to 25,940 m/d.

Gravel with a grain size of 2 - 64 mm has a hydraulic conductivity of $5*\ 10^2$ m/d to $1*10^4$ m/d.

Coarse gravel with a hydraulic conductivity of $1*10^4$ m/d does not meet the required permeability of approximately $2.6*10^4$ m/d.

The minimum grain size and its characteristics are calculated according to equation 4.14.

$$d_{10} = \sqrt{0.3 * 100}$$

$$\frac{d_{60}}{d_{10}} < 5$$

Alternatively, rock with a grading of d_{10}= 5.5 mm and d_{60}= 27.5 mm can be used. It is recommended to prove the hydraulic conductivity in an experiment first.

Design summary

	VF Wetland	HSSF Wetland
Surface	3,848 m^2	600 m^2
Filter volume	7,697 m^3	600 m^3
Diameter filter	70m	-
Length filter	-	0.5m + 5m + 0.5m
Width filter	-	100m
Depth	2 m + 0.5 m	1 m
Material	Gravel	Gravel/rock
k_f value material	5* 10^2 m/d to 1*10^4 m/d	≥26*10^3 m/d
Diffuser structure	Rotating two arm diffuser	Diffuser pipe

Table 6-1 Design summary of Option 1

A rotation diffuser could be build according to the requirements of a trickling filter diffuser system (equation 4.13).

The maximum spraying power is recommended with 4mm/arm for a four-arm diffuser system. The hydraulic surface load is 0.056 m/h.

$$a = \frac{q_{A,max} * 1000}{S_K * n}$$

$$a = \frac{0.056\ \frac{m}{h} * 1000}{4\frac{mm}{arm} * 4\ arms}$$

The diffuser system would rotate 3.5 times per hour.

It must be considered if wetland plants would be practicable for this type of design. It can be expected that in case of such a high rate system the benefit of planted media is minimal. It is not recommended to plant the vertical flow wetland as the diffuser system irrigates higher loads of water over the surface and plants could be damaged or organic litter of the plants could be removed and cause blockage of the filter media. The horizontal flow wetland could be planted with a wide variety of plants, but more for aesthetic than practicable reasons, to fulfil the natural design idea.

Performance review

Hybrid wetlands have shown great results in literature, as results for hybrid wetlands using vertical and horizontal flow wetlands listed in 4.4.5 shows. As mentioned earlier, most studies refer to pilot scale or lab scale wetlands. Case studies about build wetlands which are similar to Wairoa's conditions are hardly found. The main difference between the designs are the hydraulic designs. Wetlands build in New Zealand are mainly used for polishing the effluent. The New Zealand National Institute for Water and Atmospheric Research recommends gravel as filter media, wetlands build in Europe or the USA are using finer materials such as sand. These different recommendations lead to significant differences when considering the hydraulic design.

6.2. Option 2

As second option, an interpond rock filter application and the installation of floating wetlands is suggested.

With a total oxidation pond volume of approximately 5,350 m^3 and a median pond inflow of 2,284 m^3 per day, the hydraulic retention time is significant lower than DWA recommendations of 5 days. For the suggested Option 2, the current oxidation pond could be used instead as a sedimentation pond, whereas the second pond (current maturation pond) with a significant greater volume could be used as oxidation pond, separated in different zones with aeration, floating wetlands and for maturation. The different zones could be separated by using rock filter. Rock filters provide additional surface that allows microorganisms to grow on. Furthermore, rock filters are known for the ability to reduce suspended solids. A design schematic of how the system could look like is shown in figure 6-5.

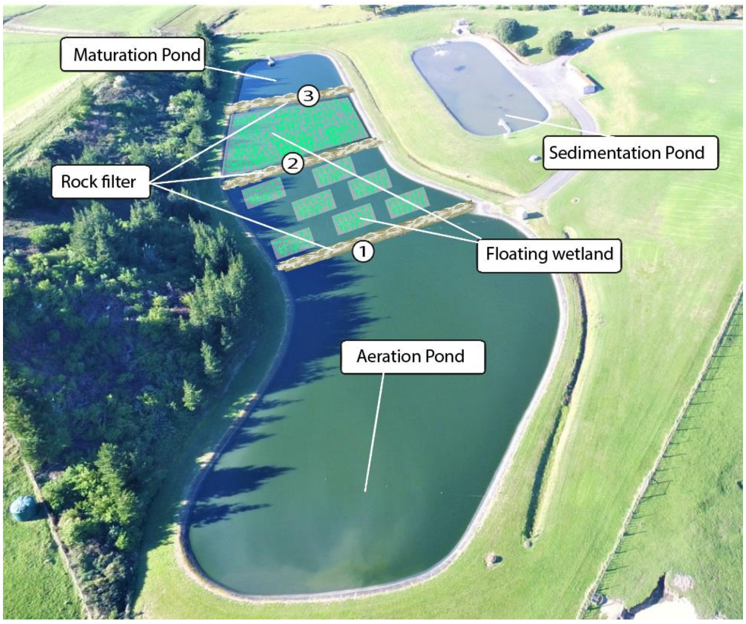

Figure 6-5 Sketch of the pond design (own representation)

As follows, a scheme of the system (figure 6-6) and a detailed description will be presented.

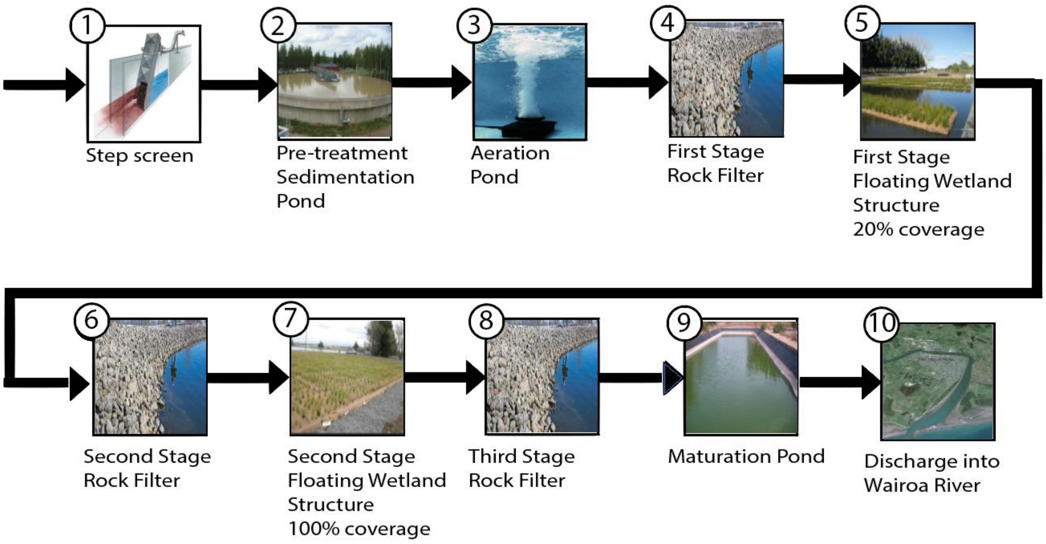

Figure 6-6 Flow scheme of Option 2 (own graphic)

Water enters the system through a step screen (1) and flows into the pre-treatment sedimentation pond (2), where settable solids settle down and pumps desludge the water mechanically. Therefore, the current aeration pond can be used. Subsequently, water runs into the aeration pond (3), which is converted from the current maturation pond. Mechanical aeration and nitrification occurs.

In step 4, 6 and 8, rockfilters separate the different zones into aeration, floating wetland aerobic and floating wetland anaerobic. The first stage floating wetland structure (5) has a coverage of approximately 20 %, which provides conditions for further nitrification. Step 7, the second stage floating wetland structure, 100 % coverage, provides conditions for denitrification. Denitrification is predominately limited by Carbon. Additional Carbon can be supplied artificially to improve denitrification. After passing the rockfilters and various wetlands, water enters the maturation pond (9), which is a polishing effluent. Afterwards, water can be discharged into the river (10).

Assessment of sedimentation pond

According to DWA worksheet A 201 sedimentation ponds should have a minimum volume of 0.5 m^3 per connected resident. The hydraulic retention time should be minimum one day during dry weather flow.

The required volume for a sedimentation pond for Wairoa would be calculated as follows.

$$V_{sedi} = 0.5m^3 * 4250\ residents$$

The calculation shows a minimum required sedimentation pond volume of 2,125 m^3, this would not meet the required hydraulic retention time of one day during dry weather flow. According to this the volume must be minimum 2,284 m^3. According to DIN EN 12255-8 it must be separated in two separate ponds (DWA, 2005). Then, the total volume would be 4,568 m^3. This is still less than the maximum available volume of 5,450 m^3. The oxidation pond would meet the minimum requirements for a sedimentation.

Assessment of a new structured treatment pond

As mentioned in DWA worksheet A 201, an oxidation pond should have a minimum hydraulic retention rate of 5 days for dry weather flows.

Related to the inflow assessment of inflow volumes for the years 2016 until present the median inflow volume is 2,284 m^3/d.

The hydraulic retention time is calculated with equation 4.2. To determine the minimum required volume, the following equation shall be used.

$$Vmin = 5d * 2{,}284 \frac{m^3}{d}$$

The minimum required volume for an oxidation pond would be 11,420 m^3.

The current maturation pond presents a total surface of 10,970 m^2 with an average depth of 2.5 m. As mentioned in 5.1.3, the maximum volume of the maturation pond is approximately 24,130 m^3. The hydraulic retention time of the whole pond is approximately 10 days.

$$t_R = \frac{24{,}130\ m^3}{2{,}284\ \frac{m^3}{d}}$$

It would be possible to divide the pond in an aerated and an unaerated area.

$$V_{min} = 2{,}284 \frac{m^3}{d} * 5d$$

The minimum required volume to meet the hydraulic retention time for Wairoa's inflow characteristics must be minimum 11,410 m^3. The pond has an average depth of 2.5 m. Thus, the minimum required surface area for the aeration pond is 4,564 m^2, 41.6 % of the total surface area.

The new designed maturation pond is mainly applied to polish the discharge of the biological treatment. A hydraulic retention time of one day is recommended. The minimum volume would be 2,284 m^3. It is expected that the depth at the end, where the new maturation pond is applied, is less than the total pond depth of 2.5 m.
Hence, a minimum depth of 2 m is assumed for the surface calculation. The minimum required surface area is calculated as 1,142 m^2. Three rock filters are applied to separate the different zones. Each rock filter has a length of one metre. The width and depth of each filter is varying, an estimated filter surface area of 116 m^2 is suggested. While filter one and two are designed to be approximately 42 m wide, the third filter is designed with only 32 m width.

The remaining surface area of 5,148 m^2 is then divided into two zones with the same size. The first zone, which Is located between aeration pond and rock filter no. 2 and is suggested to be a low coverage floating wetland, which should provide further nitrification potential for water discharged from the aeration pond. The surface area is about 2,574 m^2. The first stage floating wetland pond is estimated with a coverage of 20 %, which would be 514.8 m^2 of wetland cover. Afterwards water passes the next zone (second floating wetland), which is fully covered with wetland plants to provide anaerobic treatment before entering the maturation pond. The second zone is covered completely with wetlands. A selection of plants should be trailed before establishing the full-scale wetland. Wetland plants used for floating wetlands should not grow tall. Short wetland plants, such as bulrush are commonly used.

To select the rock filter material the flow velocity must be determined.

$$u = \frac{Q_d}{A} \qquad (6.5)$$

$$u = \frac{2284 \ \frac{m^3}{d}}{105 m^2}$$

The calculation shows a velocity of 21.73 m/d, which is equal to 0.00025 m/s. The selected material should have a hydraulic conductivity by the power of ten times more than the calculated value.

The filter material's hydraulic conductivity is recommended with minimum 0.0025 m/s.

Assessment of rock filter

Figure 6-7 shows the expected flow profile through the pond. Each rock filter has an hydraulic resistance. Based on the material, it is expected that water backs up in each pond. The material must provide enough hydraulic conductivity to avoid short circulating and a back up of the pond for higher flows as this could cause overflows. Aditionally, the hydraulic conductivity should be low enough to provide enough retention time between water and filter material.

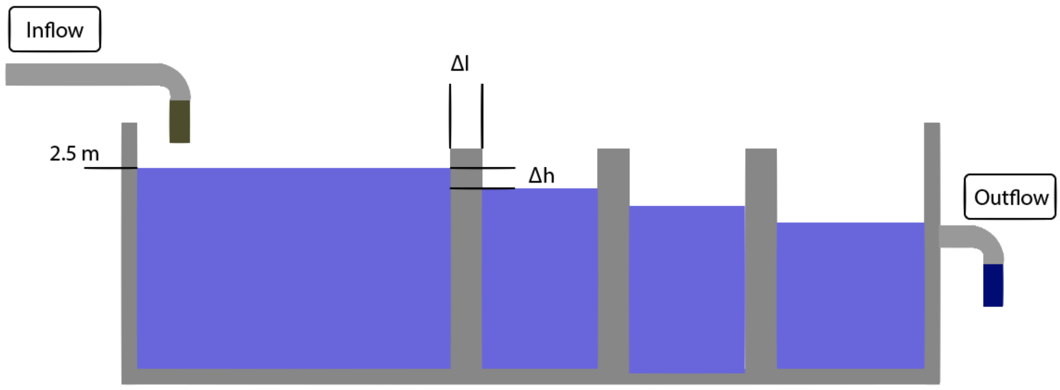

Figure 6-7 Flow profile of Option 2 (own graphic)

Filter material assessment

The main factor for selecting filter material is the capability of handling and maintaining the hydraulic head during higher inflows into the system. It is important to provide sufficient hydraulic conductivity to avoid overflows. To determine the right filter material, first assumptions about filter material were made and then proofed if it provides sufficient hydraulic conductivity. The first assumption was using gravel with a grain size of 2mm - 62mm with a hydraulic conductivity of $5*10^2$ m/d to $1*10^5$ m/d.

The calculation is based on the law of Darcy. Equation 4.13 and equation 4.14 are used to determine the hydraulic head for each filter. The combination of both equation describes the relation between inflow, hydraulic conductivity and filter length, and the resulting hydraulic head.

$$\Delta h = \frac{Q_f * \Delta l}{k_f * A_f} \qquad (6.6)$$

The material is proved according to the highest inflow of 60 l/s. Gravel with a hydraulic conductivity of 500 to 10,000 m/d is selected as material usage. The lower value is chosen to prove the hydraulic head. Flow through the filter depends on the daily volume and filter face area. The first filter is designed with a depth of 2.5 m, a width of 42 m and a length of one m, therefore the face area is 105 m^2.

$$\Delta h = \frac{5184 \frac{m^3}{d} * 1m}{500 \frac{m}{d} * 105m^2}$$

As calculated, the hydraulic head of filter two is 98 mm.

The head between the ponds will be 98 mm, which comes to 2.402 m. With a width of 42 m, a length of one m and a face area of 100.9 m^2, the hydraulic head of filter two is calculated as follows:

$$\Delta h = \frac{5184 \frac{m^3}{d} * 1m}{500 \frac{m}{d} * 100{,}9m^2}$$

The calculation shows a hydraulic head of 10.28 cm for filter two. The resulting depth of pond three is 2.3 m.
Measurements of filter three are 32 m width, 1 m long, face area 73.57 m^2.

$$\Delta h = \frac{5184 \frac{m^3}{d} * 1m}{500 \frac{m}{d} * 73.57m^2}$$

The hydraulic head of filter three is 14.1 cm. Based on the smaller face surface, the hydraulic head is greater. Resulting, the maturation pond's depth is 2.16 m.

As a maturation pond level of 2 m was assumed before, it is lower than the calculated pond level. The pond levels will variate through different flows, which must be considered in dimensioning the ponds and adding correction factors.

Design review	
New structured sedimentation pond	2 zone pond, volume per zone 2725 m^3
New structured aeration pond	Total volume 11,420 m^3, approx. depth 2.5 m, surface 4,564 m^2
First zone floating wetlands (20% coverage)	Covered surface area 514.8 m^2
Second zone floating wetland (100 % coverage)	Covered surface area 2,574 m^2
New structured maturation pond	Total volume 2,284 m^3, approx. depth 2 m, surface area 1,142 m^2
Design flow	2,248 m^3/d
Maximum flowrate	60 l/s, greater flows 60 l/s bypass

Figure 6-8 Design review Option 2

Performance review

Van Acker et al. (2005) analysed a floating treatment system for combined sewer overflows in Belgium. The system was designed to deal with variable, event driven nature of combined sewer overflows. The system contains a sedimentation pond for primary treatment. Water then flows through a long basin, almost fully covered with wetlands. Preliminary performance data showed removal of 33-68% COD, 66-95% for TSS, and 24-61% TP, but variable TN removal. The full coverage caused a lack of oxygen. The Horowhenua District Council upgraded the Shannon wastewater treatment plant with floating wetlands. A population of 2,100 residences is connected to the treatment plant, the average DWF is about 540 m^3/d. This is about double the population of Wairoa but four times the DWF. A comparison between influent end effluent quality has shown an average removal of BOD from 163 mg/L to 20 mg/L (87.7 % reduction), TSS from 208 mg/L to 37 (82.21% reduction), TKN from 44 mg/L to 16 mg/L (63.63 % reduction) and NH_4-N from 32 mg/L to 16 mg/L (50 % reduction) (Waterclean Technologies, 2015). Rangitikei District Council applied floating wetlands at the Marton wastewater treatment plant. The flowrate of 3,000 m^3/d is similar to Wairoa's daily inflow. Marton's wastewater treatment plant suffers under short hydraulic retention times 3 – 3.5 days which is again similar to Wairoa. The BOD concentration of 450 mg/L is significantly higher than Wairoa's BOD influent concentration (median 78 mg/L). Marton achieved a BOD reduction of 81 % with a Wetland coverage of 2,770 m^2 (Floating Island International, 2011).
Kauri Park nurseries, a New Zealand company that is specialized on nursing wetland plants and constructing floating treatment wetlands, estimates a removal of approximately 73 kg nitrogen and 37 kg phosphorus per year for a surface of 100 m^3 floating treatment wetlands. This would equal a reduction of 2,255 kg nitrogen per year or 6.18 kg per day.

The daily amount of nitrogen is approximately 51.58 kg nitrogen per day. A reduction of 6.18 kg per day would equal approx. 12% reduction of the total nitrogen based on Floating wetlands.

Floating wetlands have shown good results. Removal rates can be expected to be approximately 80 % for BOD, 60 – 90 % for suspended solids. Removal rates for Ammonia and Nitrogen are system specific and highly depend on the conditions of the treatment system, removal rates greater 50 % can be estimated. Furthermore, Option 2 contains rock filters which are particularly known for good TSS removal. It is estimated that Option 2 particularly decrease total suspended solids and Nitrogen.

6.3. Option 3

A trickling rock filter designed like a rotating trickling filter could be another option for Wairoa's treatment plant upgrade.

Trickling filters are not considered to be natural treatment systems, and their design is more industrial than natural. Nevertheless, they provide contact between water, air and earth media, which is the most significant parameter for cultural requirements. Many councils in New Zealand have upgraded their wastewater treatment plants with trickling filters.

Furthermore, trickling filters require less space than wetlands, which is the main restriction for Wairoa's treatment plant upgrade.

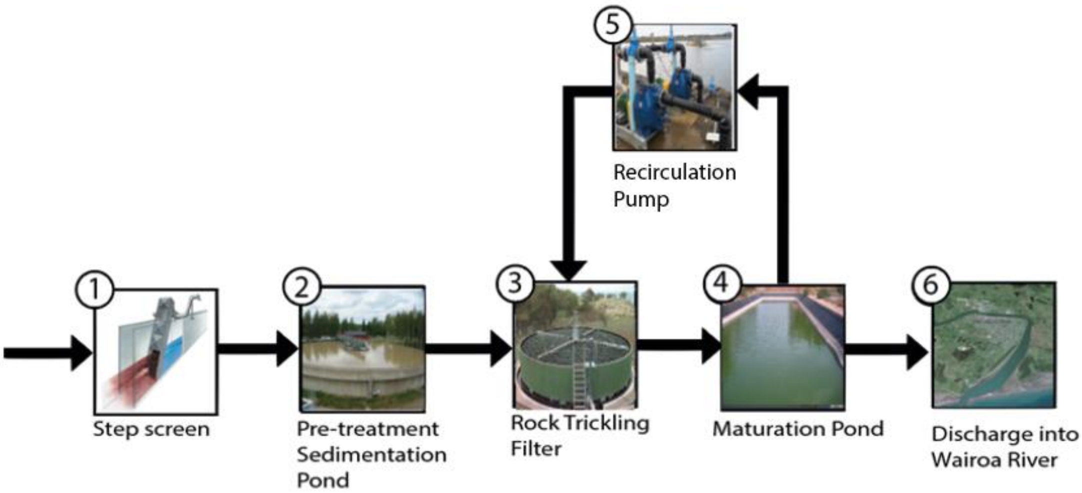

Figure 6-9 Flow scheme of Option 3 (own graphic)

Similar to Option 1 and 2, water enters the system through a step screen (1), settling in the pre-treatment sedimentation pond (2) for rough purification. From step 2, water is pumped uphill to the trickling filter (3). After that, it runs into the maturation pond (4). From there, water is pumped back to the trickling filter by a recirculation pond (5) for another treatment process. Subsequently, water that passed the system, can be discharged into the river (6).

The trickling filter calculation has been done for a two-arm distributer system, dimensioned for 60 l/s. The following design flowrates and primary effluent wastewater characteristics are determined. Rock packing with a specific surface area of 50 m^2/m^3 is recommended as filter media.

Parameter	Unit	Primary effluent
Maximum flow	m^3/d	5,184
Median flow	m^3/d	2,284
BOD	g/m^3	151
TKN	g/m^3	38

Table 6-2 Inflow parameters

The filter is designed to provide BOD_5 and Nitrogen removal. The total volume arises out of partial volumes of BOD degradation and nitrification.

For a trickling filter with rock packing, a depth of 4 m is recommended. The required surface depends on the inflow volume, the recirculation rate and the maximum surface load.

Daily volumetric load for BOD_5-removal should not exceed $B_{R,BOD} \leq 0.4$ kg/(m^3/d) and for $B_{R,TKN} \leq 0.1$ kg/(m^3/d). The required volume is calculated with equation 4.6, equation 4.7 and equation 4.8. Daily BOD_5 load, $B_{,d,BOD,ZB}$ is determined with the 85th percentile value for the BOD_5 inflow load and the median daily inflow volume.

$$V_{TK,C} = \frac{150.95\,\frac{g}{m^3} * 2284 m^3}{0.4\,\frac{kg}{m^3 * d}} \qquad [m^3]$$

As the calculation shows, the minimum required volume for BOD-removal is 862 m^3. Under consideration of nitrification, the reactor volume for nitrification is calculated as follows.

$$V_{TK,N} = \frac{38\,\frac{g}{m^3} * 2284 m^3}{0.1\,\frac{kg}{m^3 * d}} [m^3]$$

The required reactor volume for nitrification equals 868 m^3.

To calculate the total volume for trickling filter with nitrification $V_{TK,C}$ and $V_{TK,N}$ needs to be summed up.

$$V_{TK} = 862\, m^3 + 868\, m^3 \quad [m^3]$$

The total reactor volume is 1,730 m^3.

Percentile 85 of the BOD_5-inflow concentration is 151 mg/l, which almost matches the required 150 mg/l. The required recirculation is calculated with equation 4.9.

$$RV_t \geq \left(\frac{151\frac{mg}{l}}{150\frac{mg}{l}}\right) - 1$$

A minimum recirculation rate of 0.07 is required to meet the maximum inflow concentration.

Equation 4.10 is used to calculate the maximum daily inflow.

$$Q_{TK} = 2{,}284m^3 * (1 + 0.07)$$

As the calculation equals, the inflow volume is 2300m^3.

A height of 4 m is suggested for filter.

The filter surface is calculated with equation 4.12.

$$A_{TK} = \frac{1{,}730\ m^3}{4\ m}$$

Consequently, the filter surface area is 432.5 m^2.

The surface load should be in a range of 0.4 m/h to 0.8 m/h and is calculated with equation 4.11.

$$q_{A,TK} = \frac{2{,}300\frac{m^3}{d}}{24h * 432.5m^2}$$

The surface load is 0.22 m/h, the surface load is less than the minimum 0.4 m/h. The recirculation rate must be higher for dry weather flows. To determinate the minimum recirculation rate equation 4.9 is used.

$$RV_t = \left(\frac{432.5\ m^2 * 0.4\frac{m}{h} * 24h}{2284m^3}\right) - 1$$

To achieve a minimum surface load of 0.4 m/h, with daily inflow volume of 2,284 m^3 the recirculation must be RV_t =0.88. In table 6-3, the minimum recirculation rate to meet the minimum surface load of 0.4 mm/h is presented.

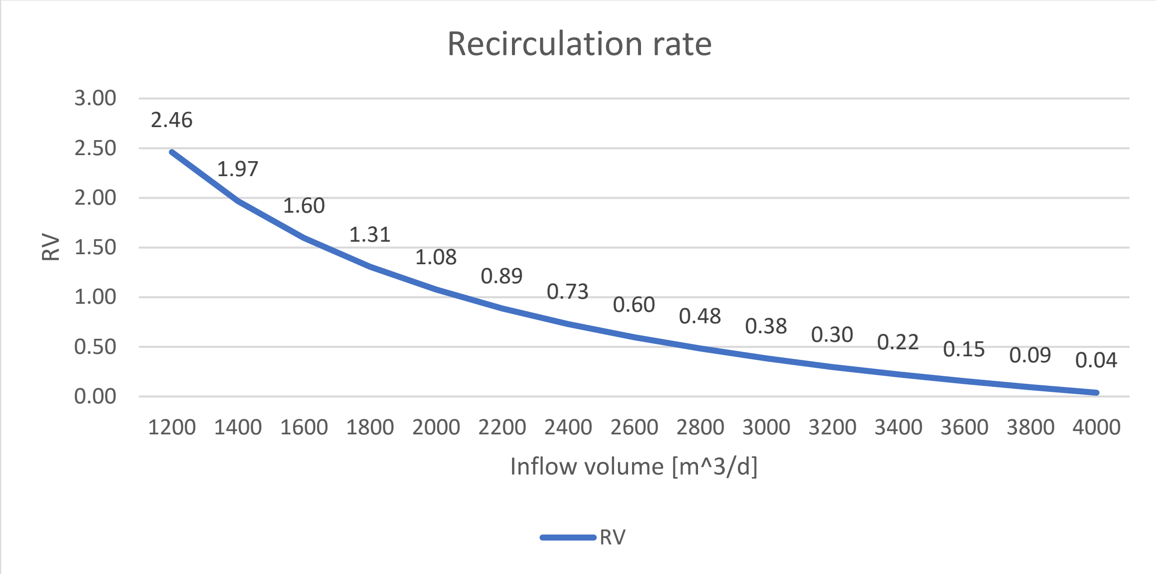

Table 6-3 Recirculation rates for different inflow volumes

The design must handle a maximum inflow rate of 60 l/s, whereas the hydraulic surface load should not exceed 0.8 m/h.

$$q_{A,max} = \frac{0.06\frac{m^3}{s} * 3{,}600s}{432.5m^2}$$

The surface load at a maximum flow rate of 60 l/s would be 0.5 m/h.

An even distribution of water over the filter is important for its operation. Spray nuzzles have to provide enough power to spread the water evenly over the filter. The spray power is calculated as follows. Where $q_{A,TK} = q_{max}$ and $q_{A,\,min} = 0.4$ m/h.

S_k is recommended to be in the range of 4 to 8 mm/arm, the rotation frequency is calculated with equation 4.13.

$$a = \frac{q_{A,min} * 1000}{S_K * n} \qquad [\text{mm/arm}]$$

$$a = \frac{0.4\,\frac{m}{h} * 1000}{4\frac{mm}{arm} * 2\ arms}$$

To maintain the minimum spraying power at a minimum surface load of 0.4 m/h the rotator should rotate maximum 50 times per hour.

$$S_{K,max} = \frac{q_{A,max} * 1000}{a * n}$$

$$S_{K,max} = \frac{0.5\frac{m}{h} * 1000}{50 * 2}$$

The spraying power at a maximum surface load of 0.5 m/h, would be 5mm/arm.

The rotation time of 1.2 minutes would be suitable to meet the recommended parameters of a spraying power of 4 to 8 mm/arm.

Performance review

Trickling filters mechanics are not fully understood yet, there is a general lack of mechanical mathematical models and design approaches. Design and operation of trickling filter processes are empirical. Because of hydraulic advantages compared to rock, commonly used filter material is synthetic media. However, trickling filters with rock media are capable of meeting treatment objectives and produce high quality effluent. It is suggested that for lower organic loads, less than 1 kg BOD_5 /d/m^3, rock media trickling filters are capable to provide the same level of treatment like trickling filters with synthetic media (Daigger & Boltz, 2011).

Performance parameters for rock trickling filters have been summarised by Daigger & Boltz (2011) and are listed below.

Design Parameter	**Carbon Oxidation and Nitrification**	**Design Parameters or Wairoa**
Media Typical Used	Rock, cross flow, or vertical flow	Rock, vertical flow
Wastewater Source	Primary effluent	Secondary effluent
Hydraulic loading [m^3/d*m^2]	14.7-88.0	11.99
BOD_5 Load [kg/ m^3*d]	0.08-0.24	0.0785
NH_3-N Load [kg/ m^2*d]	0.2 – 1.0	0.037
Effluent Concentration	< 10 mg/L as $cBOD_5$ < 3 mg/L as NH_3-N	
Depth [m]	< 12.2 m	4 m

Table 6-4 Design parameter for rock trickling filters

Expected inflow concentrations for Wairoa are lower than listed performance parameters summarised by Daigger & Boltz. Rock media trickling filters are known to provide good treatment for lower inflow concentrations.

The American Environmental Protection Agency (EPA), published a review about trickling filters that includes rock media trickling filters. The study shows great results for rock media, reduction rates for BOD_5 are reported in a range of up to 90 %, Ammonia removal in the range greater than 60 %, single study reported great removal of total suspended solids greater 90 % (EPA, 1991).

A detailed performance suggestion for Wairoa´s trickling filter cannot be made as treatment depends on many complex relations, but it can be expected that the design will show improvements regarding BOD_5, Ammonia and suspended solids. Results are listed in Appendix I.

7. Conclusion

The aim of this thesis was to develop upgrade options for Wairoa's wastewater treatment plant to meet environmental and cultural requirements. For New Zealand's native people, Maori, the spiritual relation to all things is described as Mauri. Wastewater has an exhausted or damaged Mauri, which can only be restored as the water passes through the earth and into the sea. This reflects the idea that water can be cleaned of many pollutants by passing through vegetation and the earth before entering the sea. To restore spiritual dimension, water must pass through the earth.

The main cultural attempt of each design option is the contact between water and Papatunaku, therefore each option uses some kind of natural media where water passes through. Option 1 implements this requirement with a combination of a vertical flow and horizontal subsurface flow wetland. It could be considered as the most natural option, as this option includes "rain", soakage into a media and under earth travel through the media. Because the sub-surface flow units involve effluent treatment via flow through a porous 'soil' granular medium, some (but not all) Maori accept that this meets their cultural objectives in handling human waste via 'soil' treatment before the resulting water flow enters natural water.

An application for a new discharge consent is a long and complex process. It was expected that discharge parameters for the new discharge consent would be presented to Wairoa District Council earlier this year. Unfortunately, requirements for discharge quality are still unknown. Discharge limits in New Zealand are mainly based on justifications of impacts on the receiving environment. Due to the survey regarding to the current discharge, it has been proved that even the current discharge has no significant negative impact on the river health. Nevertheless, the aim of each option is also to improve effluent quality.

During the literature research for wetlands and construction guidelines, major oppositions have been found. The wetland construction guidelines consider wetlands mainly as primary or secondary treatment. Wetlands in New Zealand are mostly applied for polishing effluent as a tertiary treatment step, used as an environmental buffer treatment stage placed between the main treatment system and a receiving water.

Main design differences can be found in hydraulic loads. In New Zealand's guidelines, it is recommended to use coarse material such as gravel, while german or/and other international guidebooks consider finer material like sand.

The main issue of a wetland design for Wairoa is limited space. Design assumptions for constructed wetlands based on German guidelines would require approximately nine times more space than available.

Therefore, alternative options which could be built on the available area have been created to meet cultural and hydraulic requirements.

Option review

Option 1

Option 1 is considered to represent the natural water cycle, as it includes natural aspects like "rain", "soakage into the soil", "water travels below the earth ", "different kinds of wetland plants" and "water meets light at the spring". All these aspects are significant steps in Maori beliefs.

The wetland design is based on required hydraulics only. The retention time is significantly low compared to conventional designed wetlands. Assumptions about nutrient removal can only be expected as this system is unique and cannot be compared or found in literature. However, wetland studies are often microcosm studies or studies about smaller scale wetlands for smaller communities. Studies about bigger scale wetlands are rare in the literature. It is expected that construction works and costs for option 1 are more elaborately than option 2 and 3. Operation costs are mainly caused by pumping costs and maintenance of the wetlands.

Option 2

Option 2 considers floating wetlands and interpond rock filters, as an easy construction and cheap upgrade option. Inter pond rock filters have the advantage, that biomass grows on the rocks, which binds algae and floating biomass. They act as artificial growth media and can assist with nitrification and especially denitrification. Rock filters provide contact between earth media and water. It is a common used and inexpensive upgrade method in New Zealand. Even though, it is not clear if rock filters are sufficient enough to address and satisfy cultural requirements.

Additionally, floating wetlands are used to improve biological treatment. In two zones, conditions for nitrification and denitrification are created to remove nitrogen. Additional Carbon can be added to provide a more efficient denitrification. Carbon could be added artificially through e.g. glucose or $CaCO_3^-$. It could be considered to shift the denitrification zone and feed the anoxic basin with raw effluent, as this could be cheaper to operate.

Studies or available data about the success of this kind of natural treatment process is limited. Insufficient data is available on long-term operation and maintenance. It is expected that a lack of knowledge and insufficient maintenance caused failure of floating wetlands in New Zealand.

Construction costs are expected to be lower than for Option 1 and 3.

Option 3

Option 3 utilises a trickling filter with rock packing as a more industrial option to realize cultural and treatment requirements. Trickling filters have been commonly used in New Zealand as upgrades for existing treatment plants. Modern trickling filters are usually built with synthetic filter material. To satisfy cultural issues, rock packing is used. Trickling rock filters have shown great results in the past and significant nutrient removal can be expected. It must be noted that trickling filter are often applied as the only biological treatment, supplemented with clarifiers only. Regarding to Wairoa's treatment plant, it would be added to the existing treatment ponds which relieves the system.

Construction costs for trickling filters in the literature are rated as high. Even though, they require less maintenance and low operation costs. Costs are mainly by electricity, used for pumping of inflow and recirculation flows.

Issues

Some issues occurred during working on this thesis, which led to a less objective result. Part of the thesis was consulting and re-consulting with local Maori about the cultural significance of each option. Due to the unreliability of individuals, this couldn't be done during predetermined time. Consequently, cultural approval for the designed options could not be reached.

Monitoring treatment process data of the existing plant is considered to be insufficient for a detailed performance review. Financial issues and the timeframe of this thesis did not allow more detailed monitoring. Existing data delivers a rough estimate of the treatment performance.

It was expected to receive novel requirements for a new discharge consent within the timeframe of the thesis, which did not arise.

Based on insufficient monitoring data, it was not practicable to dimension systems on nutrient removal rates. However, hydraulic requirements have been the restricting factor.

Additional research

During literature research, only a limited number of studies about bigger scale wetlands have been found. Wetlands have shown great success, but studies are mainly based on lab- or pilot scale systems. There is no guideline which considered wetlands as tertiary treatment for polishing of treated effluent. Guidelines are based on smaller scale constructed wetlands. Case studies about floating wetlands have shown great results but a generalised guideline with design recommendations is not available. Only a few companies in New Zealand are specialised in constructing floating wetlands.

Recommendations

Summarised all information presented in this thesis, Option 2 seems to be the most suitable option for Wairoa. A combination of inter pond rock filter and floating wetlands might not be the optimum cultural option. Even though, considering all aspects such as costs, maintenance, available land area and cultural acceptance in general, shows it is the most practicable option. It requires a minimum of construction works and is expected to be the cheapest option for Wairoa's rate payers. Floating island structures could be build and maintained indoor, as a cost-effective solution.

Furthermore, this system would be most suitable for variating flowrates, which is one of the main problems for the Wairoa Wastewater Treatment Plant.

Outlook

While theoretical research has been done and practicable approaches has been made, the next steps would be to proof financial conditions of each system. Setting up detailed financial statistics and plans is required to check if the modifications are feasible for Wairoa District Council and the ratepayers. In behalf of that, consultation with Tangata Whenua is necessary to find out if the suggested options would meet their cultural requirements.

Before building a wetland, different materials and plants should be trialed to determine if the design meets the environmental requirements of Wairoa’s treatment plant upgrade. As Option 2 is recommended, test scale floating treatment wetlands could be build to trial and identify the best suitable plant species for Wairoa. A significant focus should be placed on improvements of the reticulation system, as high inflows of rainwater into the sewer system are limiting the performance of the treatment plant. Wairoa District Council should first point out a target for reticulation improvements. In addition, estimating a maximum target flow during rain events is necessary. Improvements should further include more intensive monitoring to determine the condition of the treatment process.

However, the lack of monitoring data and invalidated flow data lead to the assumption, that Wairoa’s wastewater system has not been managed very well in the past. The upgrade of the treatment system would be a good point in time to change the management philosophy from a passive to a proactive management.

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Appendix

Appendix A

Appendix A: Available Land Area (GIS)

Appendix B

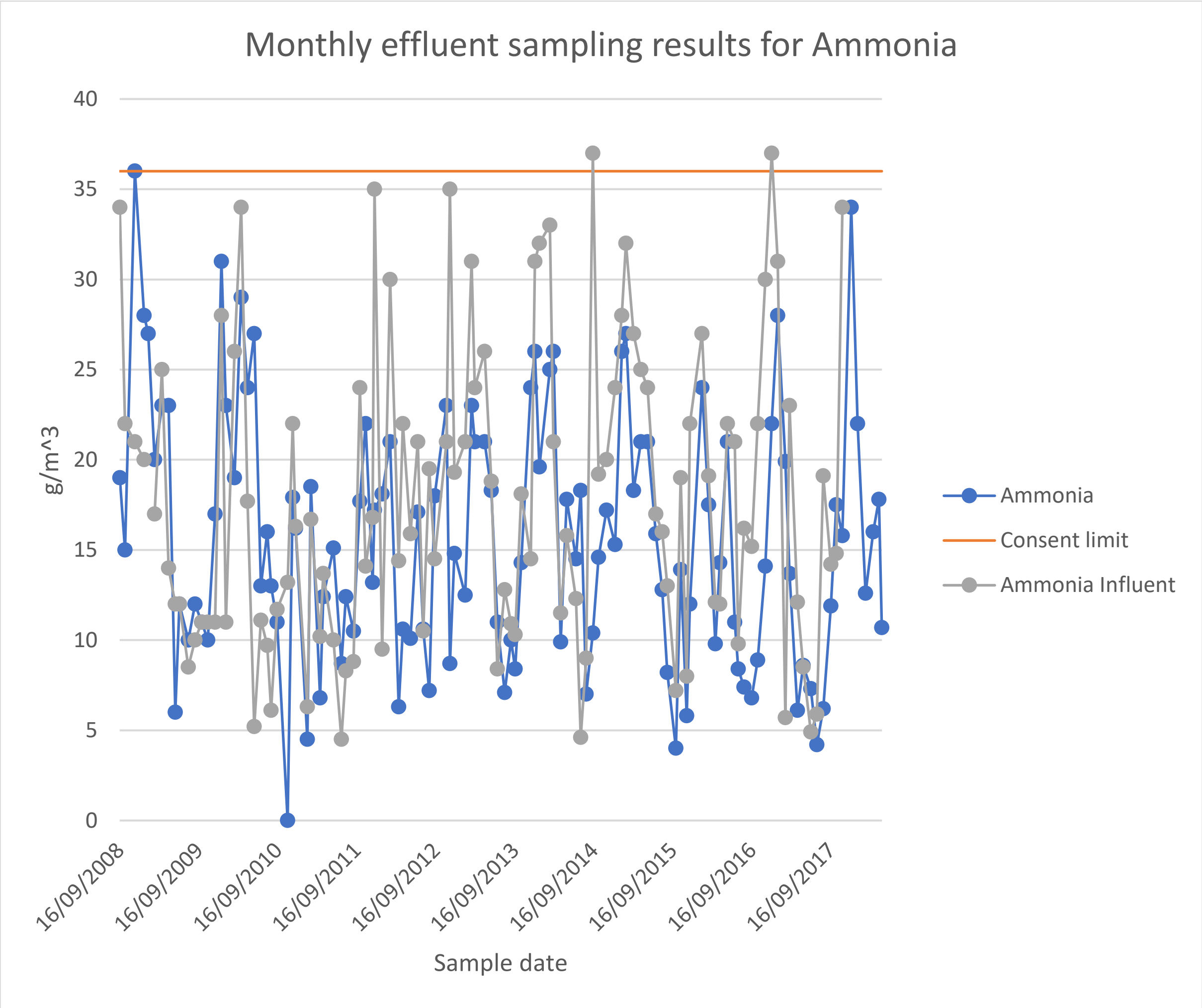
Table 1-1 Typical effluent results for one and two cell facultative WSP systems: (Hickey et al 1989)

Contaminant	Minimum	Median	95%ile
BOD_5 (mg/l)	7	27	70
Suspended solids (mg/l)	10	56	150
Faecal coliform bacteria (#/100 ml)	9×10^1	4.3×10^3	2.3×10^5
Total Phosphorus (mg/l)	1.3	8.2	11.3
Dissolved Reactive Phosphorus (mg/l)	9.5	5	0.8
Ammoniacal Nitrogen (mg/l–N)	0.001	7	29

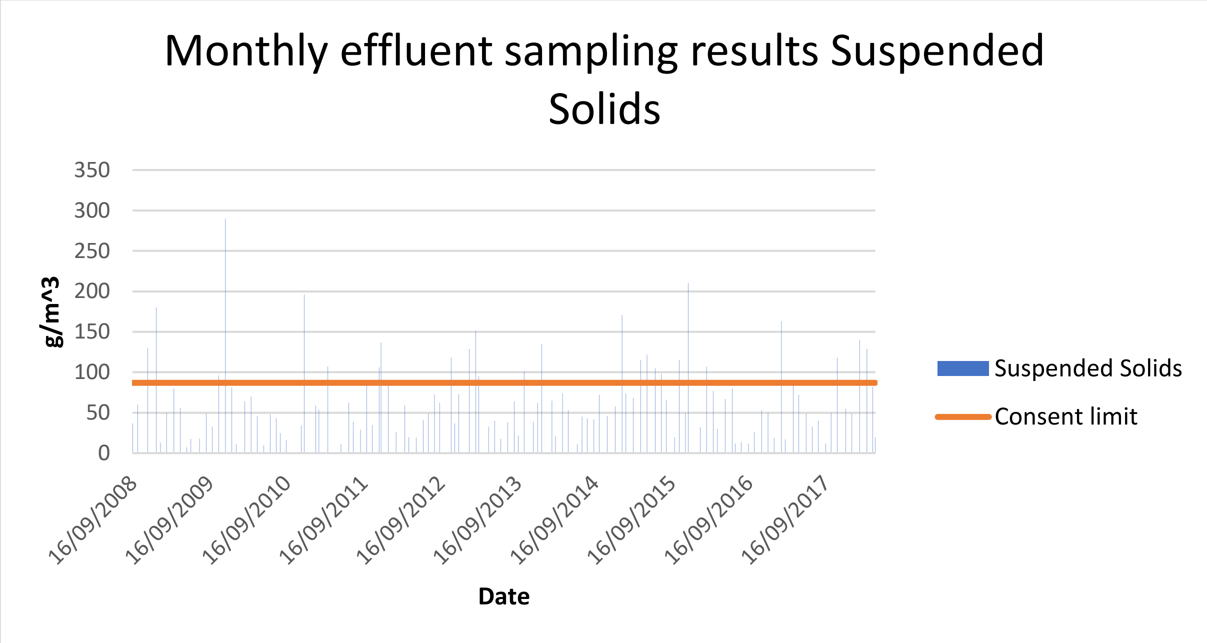
Appendix B: Typical effluent results for one and two cell facultative WSO systems (Hickey et al, 1989)

Parameter	**Outflow Wairoa WWTP**		**Typical effluent results for one and two cell facultative WSP systems**	
	Median	95th percentile	Median	95th percentile
Suspended solids	44	142.75	56	150
Ammonia	17.5	31.3	7	29
CBOD5	23	69.9	27	70
e. coli	1800	240000	4300	230000

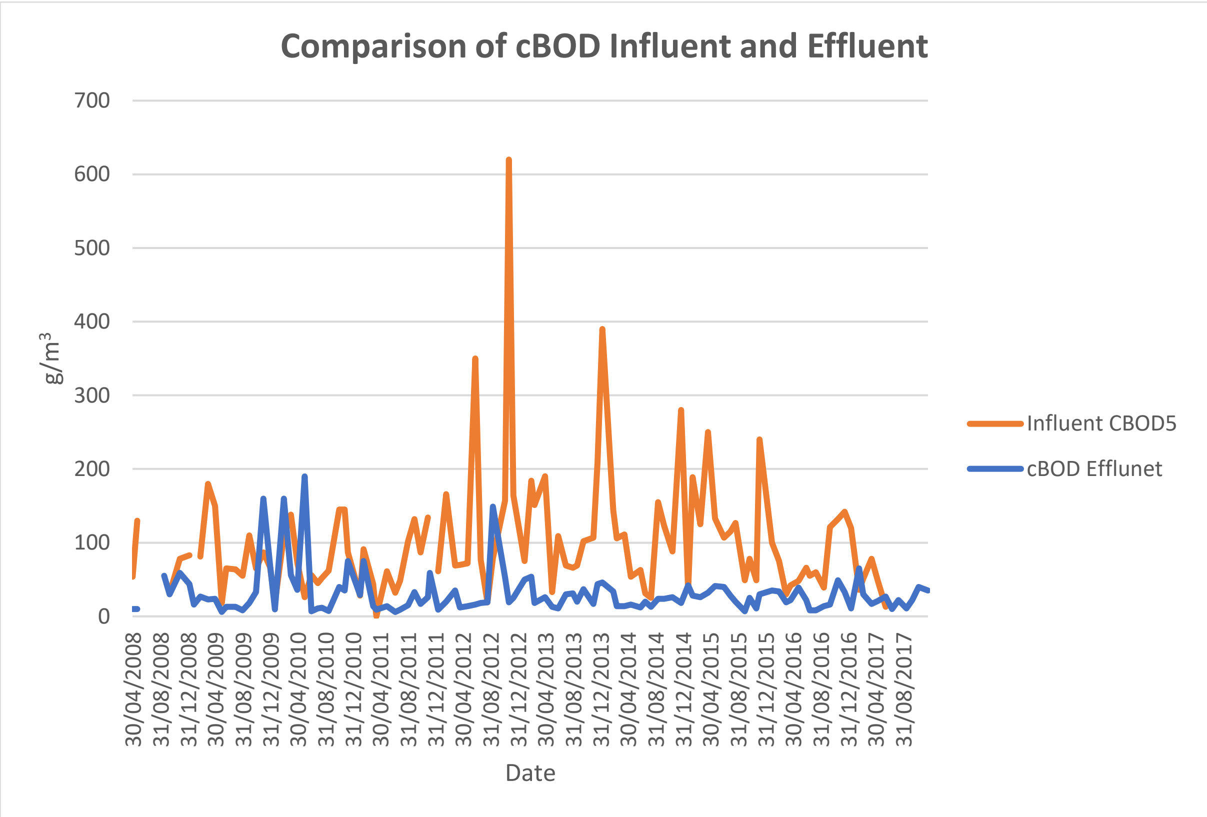
Appendix B: Comparison of Wairoa's effluent and typical effluent results for one and two cell facultative WSP systems (NZWWA, 2005)



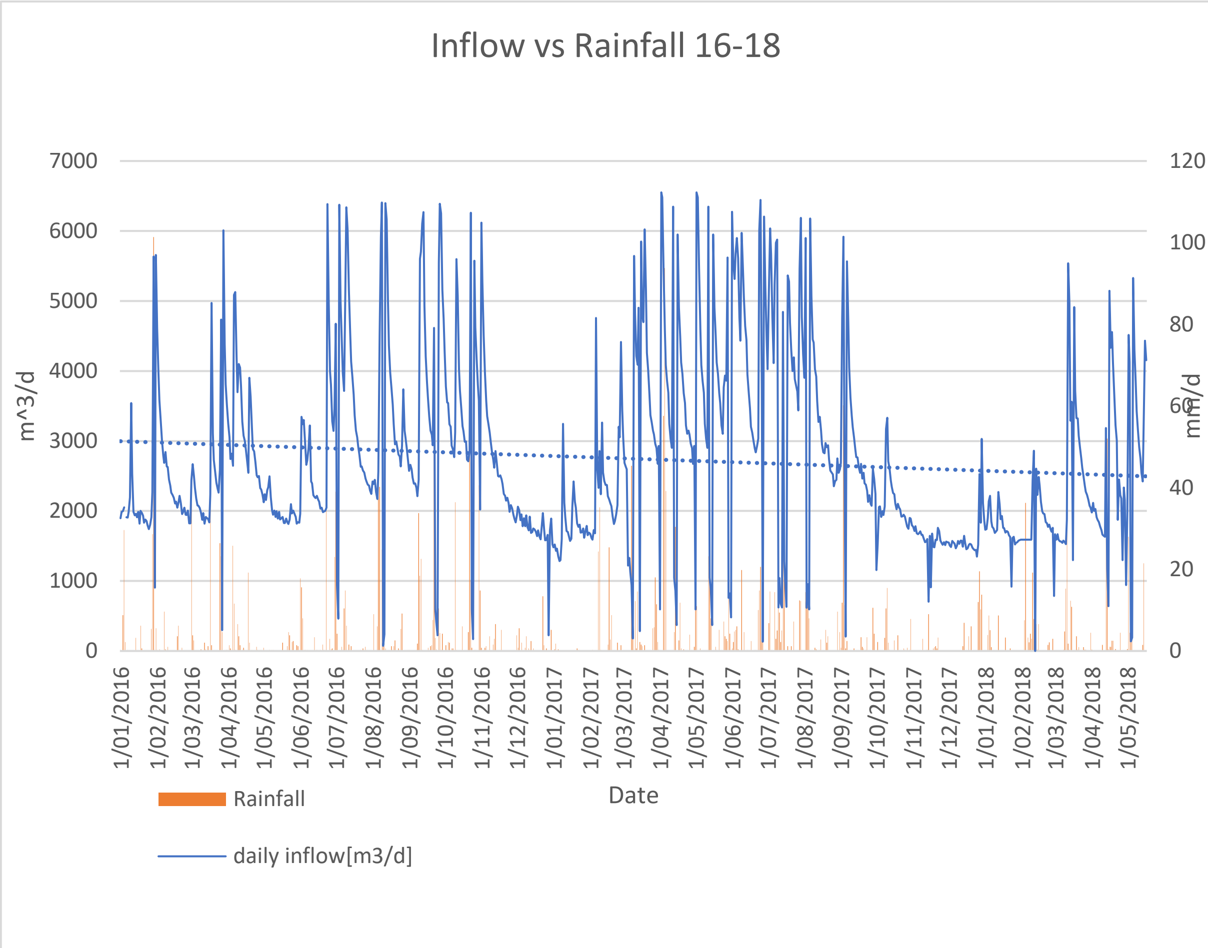
Appendix B: Monthly effluent sampling results for Ammonia



Appendix B: Monthly effluent sampling results Suspended Solids



Appendix B: Comparison of cBOD Influent and Effluent



Appendix B: Inflow vs Rainfall 16-18

1		AWF			WWF			DWF	
2	Median	2284			2634			2176	
3	Max	6552			6552			6482	
4	99th percentile	6341.81			6397			6246.85	
5	85th percentile	4116.85			5031.75			3800.75	
6	70th percentile	3042.1			3577.5			2852.1	
7	30th percentile	1873.9			2015.5			1817.5	
8	10th percentila	1527			1252			1546	
9									
10									
11		awf			wwf			dwf	
12	Date	Inflow Volume m3/d	Rain mm	Date	Inflow Vol	Rain mm	Date	Inflow Vol	Rain mm
13	1/01/2016 0:02	1910	0.2	1/01/2016 0:02		0.2	1/01/2016 0:02	1910	0.2
14	2/01/2016 0:02	1904	0	2/01/2016 0:02		0	2/01/2016 0:02	1904	0
15	3/01/2016 0:02	1990	8.8	3/01/2016 0:02	1990	8.8	3/01/2016 0:02		8.8
16	4/01/2016 0:02	2193	29.6	4/01/2016 0:02	2193	29.6	4/01/2016 0:02		29.6
17	5/01/2016 0:02	3542	2.2	5/01/2016 0:02	3542	2.2	5/01/2016 0:02		2.2
18	6/01/2016 0:02	2532	0	6/01/2016 0:02		0	6/01/2016 0:02	2532	0
19	7/01/2016 0:02	2339	0.2	7/01/2016 0:02		0.2	7/01/2016 0:02	2339	0.2
20	8/01/2016 0:02	2174	0	8/01/2016 0:02		0	8/01/2016 0:02	2174	0
21	9/01/2016 0:02	2097	0	9/01/2016 0:02		0	9/01/2016 0:02	2097	0
22	10/01/2016 0:02	2008	0	10/01/2016 0:02		0	10/01/2016 0:02	2008	0
23	11/01/2016 0:02	1930	0	11/01/2016 0:02		0	11/01/2016 0:02	1930	0
24	12/01/2016 0:02	1990	0	12/01/2016 0:02		0	12/01/2016 0:02	1990	0
25	13/01/2016 0:02	1937	0	13/01/2016 0:02		0	13/01/2016 0:02	1937	0
26	14/01/2016 0:02	1962	3.2	14/01/2016 0:02	1962	3.2	14/01/2016 0:02		3.2
27	15/01/2016 0:02	1911	0	15/01/2016 0:02		0	15/01/2016 0:02	1911	0

Appendix B: Daily inflow volume

Wairoa Daily consumption 2016/2017			Daily water consuption [m^3/d]		
Date	Daily volume to Town	Volume excluisive AFFCO (43.3 %)	Median	85th Percentile	per capita
1/01/2016	2751	1191	1198	1455	0.342386
2/01/2016	2917	1263			
3/01/2016	2708	1172			
4/01/2016	2987	1293			
5/01/2016	2567	1112			
6/01/2016	2835	1228			
7/01/2016	2872	1243			
8/01/2016	3070	1329			
9/01/2016	3187	1380			
10/01/2016	3574	1548			
11/01/2016	3475	1505			
12/01/2016	3336	1444			
13/01/2016	3504	1517			
14/01/2016	3315	1435			
15/01/2016	3181	1378			
16/01/2016	3669	1588			
17/01/2016	3870	1676			
18/01/2016	4035	1747			
19/01/2016	3285	1422			
20/01/2016	3347	1449			
21/01/2016	3923	1699			
22/01/2016	3959	1714			
23/01/2016	3893	1686			
24/01/2016	3830	1659			
25/01/2016	4166	1804			
26/01/2016	4184	1812			

Wairoa Water consumption Jan 20

Appendix B: Daily water consumption Wairoa (Wairoa District Council)

Appendix C

Wairoa District Council, resource consent monitoring.

Wairoa Sewage Treatment Plant Effluent

RESULTS 08/01/2000 - PRESENT

	ANALYSIS	UNIT	Median Discharge	85th percentile	95th percentile
EAM SAMPLE ID					
FIELD TESTS	Temperature	0C	17,3	22,88	25
	pH	pH	7,6	8,2	8,603
	Dissolved Oxygen	g/m3	6,2	12	16,52
	Conductivity	US/cm	480	643,2	791,4
	Salinity	0/00 (ppt)	0	0,2	0,4
	Turbidity	NTU	32	66,5	94
CHEMICAL	COD	g/m3 as O	110	220	323
	Ammonia	g/m3 as N	17,5	26	31,3
	Suspended Solids	g/m3	44	96,75	142,75
	cBOD	g.O2/m3	23	44,6	69,6
	Absorbance at 254nm	AU	0,266	0,3496	0,4391
	Transmittance at 254nm	%T	54	62,93	65,46
MICRO	*e.coli*	cfu/100ml	7.300	78.750	240.000
	Enterococci	cfu/100ml	1.800	13.000	44.800

Appendix C: Wairoa District Council, resource consent monitoring (Wairoa Sewage Treatment Plant Effluent)

Wairoa District Council, resource consent monitoring.

Wairoa Sewage Treatment Plant Raw Influent

	ANALYSIS	UNIT	CONSENT LIMITS	DETECTION LIMIT	median	85th	95th
EAM SAMPLE ID							
FIELD TESTS	pH	pH		0,1	7,5	8,005	8,3
	Conductivity	US/cm		1	480	609,8	714,6
	Temperature	0C		0,5	18,2	20,4	21,32
	Dissolved Oxygen	g/m3		0,1	2,95	4,535	5,1
	Salinity	0/00 (ppt)		0,1	0,1	0,2	0,3
	Turbidity	NTU			68	109,8	199,8
CHEMICAL	Total Alkalinity	g/m3 as $CaCO_3$		1	163	220	238
	Total Nitrogen	mg/l		0,05	23	38	43,8
	Ammonia Influent	g/m3 as N		0,01	16,3	27,2	34
	Nitrate + Nitrite	mg/l		0,002	0,007	0,6625	2,535
	Total Kjeldahl Nitrogen	g/m3 as N		0,1	23	38	43,8
	Total Phosphorus	g/m3 as P		0,004	3,4	5,2	6,66
	$CBOD_5$ (carbonaceous)	g/m3 as O		1	78,5	164	263,5
	COD	g/m3 as O		6	240	410	668
	Total Oil & Grease	g/m3		4	24	44	120

Appendix C: Wairoa District Council, resource consent monitoring (Wairoa Sewage Treatment Plant Influent)

Appendix D

Daily surface load [l/m^2*d]								
Inflow	m^3/d →	2000	2500	3000	3500	4000	4500	5000
Area [m^2]↓	l/s →	23	29	35	41	46	52	58
1000		2000	2500	3000	3500	4000	4500	5000
2000		1000	1250	1500	1750	2000	2250	2500
3000		667	833	1000	1167	1333	1500	1667
4000		500	625	750	875	1000	1125	1250
5000		400	500	600	700	800	900	1000
6000		333	417	500	583	667	750	833
7000		286	357	429	500	571	643	714
8000		250	313	375	438	500	563	625
9000		222	278	333	389	444	500	556
10000		200	250	300	350	400	450	500
11000		182	227	273	318	364	409	455
12000		167	208	250	292	333	375	417
13000		154	192	231	269	308	346	385
14000		143	179	214	250	286	321	357
15000		133	167	200	233	267	300	333
16000		125	156	188	219	250	281	313
17000		118	147	176	206	235	265	294
18000		111	139	167	194	222	250	278
19000		105	132	158	184	211	237	263
20000		100	125	150	175	200	225	250
21000		95	119	143	167	190	214	238
22000		91	114	136	159	182	205	227
23000		87	109	130	152	174	196	217
24000		83	104	125	146	167	188	208
25000		80	100	120	140	160	180	200
31000			81	97	113	129	145	161
37000				81	95	108	122	135
43000					81	93	105	116
50000						80	90	100
56000							80	89
64000								78

Appendix D: Calculation of the daily surface load

Appendix E

Appendix E: Sludge Survey Report for Wairoa District Council Pilot Hill Wastewater Ponds

Pilot Hill Wastewater Treatment Plant

This report has been prepared for Wairoa District Council by Parklink Ltd. No liability is accepted by this company or any employee or sub-consultant of this company with respect to its use by any other parties.

This disclaimer shall apply notwithstanding that the report may be made available to other persons for an application for permission or approval to fulfil a legal requirement.

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Sludge Survey Report
For Wairoa District
Council Pilot Hill
Wastewater Ponds

Table of Contents

Attachments

Appendices: Drawing 1176-001 Sample & Profile Locations
Drawing 1176-002 Sludge Depths
Drawing 1176-003 Long Sections

References: Indicative Proposal emailed to Council 12/04/16

1.0 Introduction

Parklink Ltd have been contracted to survey the Pilot Hill Wastewater Ponds to determine the following:

- Base profile
- Sludge interface levels
- Sludge volumes
- Sludge densities and any stratification
- Dry mass
- Inorganics

This sludge survey was carried out on Wednesday 4th May 2016 in calm weather. The volumes stated in this report are based on the water of each pond on the day.

The Pilot Hill wastewater ponds are shown

below in Figure 1.

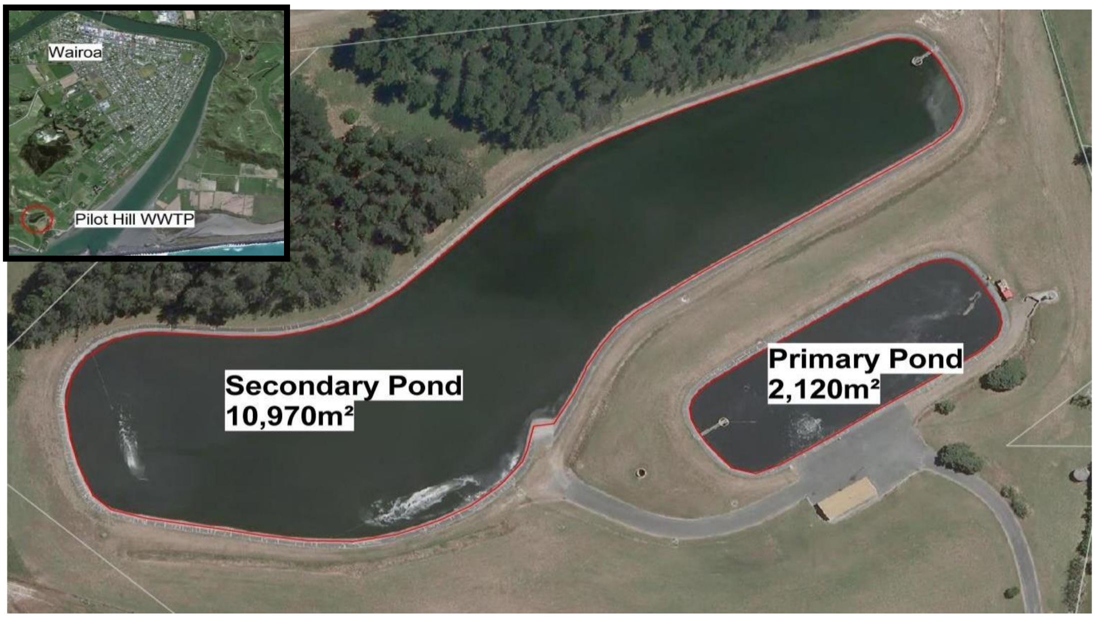


Fig. 1

2.0 Methodology

3D Geo mapping software:

Origin of levels and coordinate system are assumed local datum in terms of 3 bench marks set up on site.

Survey of the base of the ponds: The base of the ponds were surveyed with high accuracy Total Station LEICA 1203 and a staff with a prism. The survey was carried out from a boat, on a 10m by 10m grid on average. Top of wave band, batter and water level were also surveyed around the perimeter of each pond.

Survey of Sludge: The sludge levels were surveyed using the same Total Station for location and a graduated sludge judge to measure to the top of the sludge layer. This survey was also carried out from a boat on a 15m by 15m grid on average.

Drawings and Volumes: Field observations were downloaded into 12d software which was used to produce plans, contours, depths, long sections, and volumes.

Sludge Sampling:

In situ sludge samples were taken at various locations and at up to three different depths at each location to be analysed for dry solids density and volatile solids. See sample and profile location map.

Sampling of the sludge is done by inserting a graduated vacuum sampler to a specific depth which is then opened via a pneumatically operated pinch valve to allow sludge to enter the sample chamber. A vacuum pump is then activated to assist the sludge to enter the chamber and then the chamber is shut via the pinch valve. The sample is lifted and emptied into a sampling jar.

A sample of the sludge is collected at each transect. In the deeper regions up to three samples are collected per transect. Samples are sent to the laboratory to be analysed for total dry solids using the following test methodology.

-Test Methodology: Total Solids APHA 21st Ed. 2540B
-Test Methodology Detection Limit: % wt/ wet/ wt

The densities were entered into the spreadsheet alongside their respective depth and transect location. This enables us to form a reasonably accurate assessment of the relevant densities of sediment at each location and whether there is any evidence of density stratification or change in density relative to depth.

A selection of the samples were also analysed as composite samples for volatile solids using the following test methodology.

Organic Matter Calculation:	100 - Ash (dry wt).	
	0.04 g/100g dry wt 1-4	
Ash Ignition in muffle furnace 550°C, 6hr, gravimetric. 2012	APHA 2540 G 1-4	22nd ed.
	0.04 g/100g dry wt	

3.0 Primary Pond Sludge Volumes & Characteristics

The base profile varies in depth up to 3.9m below the water level on the day of the survey.

Approximate maximum total capacity of pond: **5,350 m³** (Assumed water level RL 100.10m)

Total pond volume based on water level on the day: **4,753 m³** (RL 99.82m)

Approximate wet sludge volume: **2,750m³**

Percentage of pond occupied by sludge: **57.9%**

At each survey location, samples were taken at up to three different depths to identify any change in density relative to depth which is often due to compaction. Sludge dry solids densities ranged from 7.83% to 29.68%. From the samples collected, we were able to identify an increasing density trend relative to depth however sample results around 20% dry solids could suggest the presence of inert material such as grit which has a higher specific gravity than biodegradable organic solids.

Typically when there is evidence of stratification in density, Parklink separate up to three layers of the sludge volume (upper sludge, mid and base sludge) to which we apply the appropriate average dry solids density to calculate a total dry mass.

Taking into account this stratification the following average dry solids (DS) densities can be applied to each layer.

Upper sludge average: **9.63% DS**

Mid sludge average: **15.86% DS**

Lower sludge average: **21.73% DS**

When applying these to the corresponding sludge volumes (upper, mid and base sludge) the total dry solids equates to: **377.7 m³ Total Dry Solids**

As a comparison, the *overall* average density from all samples taken, and then applied to the total sludge volume equates to: **398.8 m³ Total Dry Solids**

Overall average dry solids density of all samples taken: **14.50% DS**

A selection of the samples were also formed into a composite sample and analysed for volatile solids to determine the extent of inert or inorganic content.

Representative composite sample: **38% organic matter and 62% ash (inorganics)**

This shows a relatively high content of inert material which could be due to stormwater infiltration during high rain events.

4.0 Secondary Pond Sludge Volumes & Characteristics

The base profile varies in depth up to 2.43m below the water level on the day of the survey.

- Approximate maximum total capacity of pond: **24,130 m³** (Assumed water level RL96.65m)
- Total pond volume based on water level on the day: **18,260 m³** (RL 96.13m)
- Approximate wet sludge volume: **5,863m³**
- Percentage of pond occupied by sludge: **32.1%**

At each survey location, samples were taken at up to three different depths to identify any change in density relative to depth which is often due to compaction. Sludge dry solids densities ranged from 8.46% to 10.7%. From the samples collected, we were not able to identify an increasing density trend relative to depth.

As there was no apparent change in density relative to depth, the *overall* average density from all samples taken, was applied to the total sludge volume which equated to: **537.6 m³ Total Dry Solids**

- Overall average dry solids density of all samples taken: **9.17% DS**

A selection of the samples taken from the Northern half of the pond and likewise the Southern half and were formed into a composite samples. These composites were analysed for volatile solids to determine the extent of inert or inorganic content.

- Northern representative composite sample: **45% organic matter and 55% ash (inorganics)**
- Southern representative composite sample: **43% organic matter and 57% ash (inorganics)**

5.0 Summary & Recommendations

The primary pond has large accumulations of sludge and a relatively high component of inorganics. This pond could greatly benefit in terms of increased capacity/retention time and pre-treatment efficiency by reducing the amount of solids that have built up over time. This would in turn, lighten the load on the secondary pond which, over time has also accumulated solids as they have flowed through from the primary pond. The secondary pond has a fairly compact layer of sludge that equates to approximately 30% of the total pond volume. This pond could also increase in treatment efficiency with a reduction firstly in the dry solids content of the sludge which would in time also influence a reduction in sludge volume.

With new consents pending and the possibility of a tightening of discharge requirements, Parklink recommend Advanced Microbial Digestion (AMD) to Council as a proven and cost effective method to reduce solids accumulations and stimulate the important biological functionality that contributes to an efficient wastewater treatment system. AMD is an in-situ sludge management tool meaning the ponds don’t have to be taken off line and the environmental impact is very small.

Refer to Parklink’s Indicative Proposal dated April 2016 for more information on the advantages of AMD and the likely costs for a system at the Pilot Hill wastewater treatment plant.

Appendix F

Appendix F: Oxidation Pond desludge calculation

Calculation accoding to sludge density:

14 Sludge treatment and disposal

Table 2.3. Density, specific gravity, VS/TS ratio and percentage of dry solids for various sludge types

Types of sludge	VS/ST Ratio	% dry solids	Specific gravity of solids	Specific gravity of sludge	Density of sludge (kg/m^3)
Primary sludge	0.75–0.80	2–6	1.14–1.18	1.003–1.01	1003–1010
Secondary anaerobic sludge	0.55–0.60	3–6	1.32–1.37	1.01–1.02	1010–1020
Secondary aerobic sludge (conv. AS)	0.75–0.80	0.6–1.0	1.14–1.18	1.001	1001
Secondary aerobic sludge (ext. aer.)	0.65–0.70	0.8–1.2	1.22–1.27	1.002	1002
Stabilisation pond sludge	0.35–0.55	5–20	1.37–1.64	1.02–1.07	1020–1070
Primary thickened sludge	0.75–0.80	4–8	1.14–1.18	1.006–1.01	1006–1010
Second thickened sludge (conv. AS)	0.75–0.80	2–7	1.14–1.18	1.003–1.01	1003–1010
Second thickened sludge (ext. aer.)	0.65–0.70	2–6	1.22–1.27	1.004–1.01	1004–1010
Thickened mixed sludge	0.75–0.80	3–8	1.14–1.18	1.004–1.01	1004–1010
Digested mixed sludge	0.60–0.65	3–6	1.27–1.32	1.007–1.02	1007–1020
Dewatered sludge	0.60–0.65	20–40	1.27–1.32	1.05–1.1	1050–1100

Notes:
For specific gravity of solids use *Equation* 2.1; for specific gravity of sludge use *Equation* 2.2
AS = activated sludge; ext. aer. = extended aeration activated sludge

Values for sludge density can be found in Table above. Dewatered sludge with a dry solids content of 20% DS has a density of 1050 kg/m^3 and dewatered sludge with 40% density is 1100 kg/m^3 .

ECL Group desludged 550.3 t of sludge with an approx. dry solids content of 25% of the Wairoa wastewater treatment plant.

Sludge density is determinated through lineare interpolation.

f(x)= $f0 + \frac{f1-f0}{x1-x0} + (x - x0)$ f(x)= $1050 + \frac{1100-1050}{40-20} + (25 - 20)$

The dry solids contant for 25% DS is 1062.5 kg/m^3 = 1.0625 t/m^3.

The Volume can be calculated according to:

$$V = \frac{m}{\rho} = \frac{550.3}{1.0625}$$

Where:

ρ = specific density kg/m^3

m= mass kg

$V = m$ ECL removed approximately 518 m^3 wetsludge.

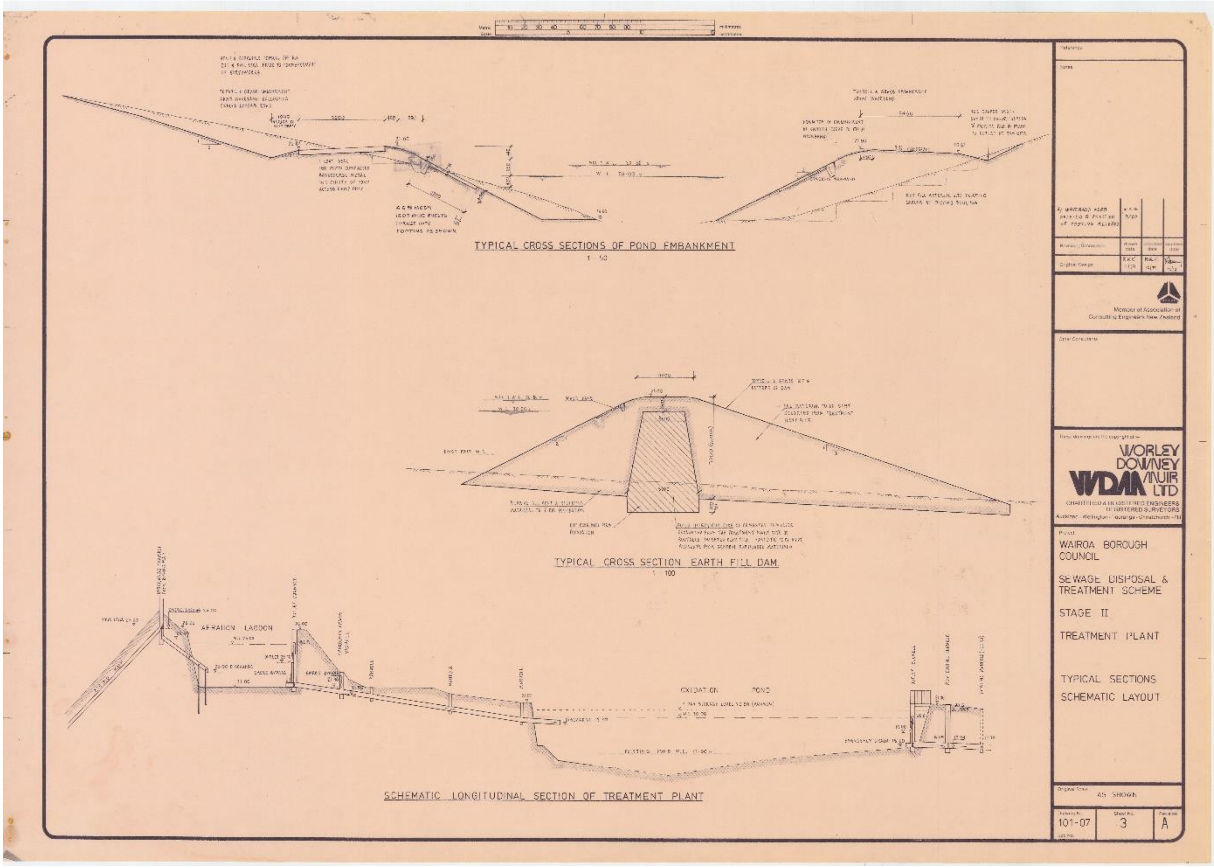
Appendix G

Table 1 Porosities and hydraulic conductivities for various rocks and sediments.

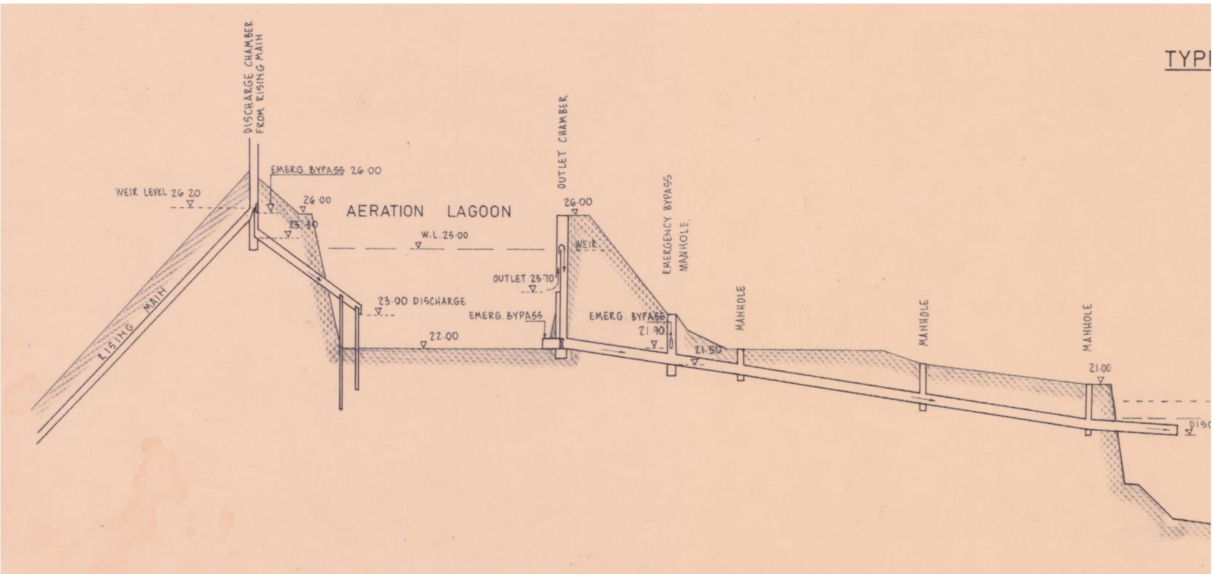
Geological material	Grain size/mm	Porosity (%)	Hydraulic conductivity/m per day
Unconsolidated sediments			
clay	0.0005 to 0.002	45 to 60	less than 10^{-2}
silt	0.002 to 0.06	40 to 50	10^{-2} to 1
sand	0.06 to 2	30 to 40	1 to 5 x 10^{2}
gravel	2 to 64	25 to 35	5 x 10^{2} to 1 x 10^{4}
Consolidated sedimentary rocks			
shale	fine	5 to 15	5 x 10^{-8} to 5 x 10^{-6}
sandstone	medium	5 to 30	10^{-4} to 10*
limestone	variable	10^{-1} to 30* (solution porosity)	10^{-5} to 10*
Igneous and metamorphic rocks			
basalt	fine	10^{-3} to 1 (up to 50 if vesicular)	3 x 10^{-4} to 3*
granite	coarse	10^{-4} to 1 (up to 10 if fractured)	3 x 10^{-4} to 1*
slate	fine	10^{-3} to 1	10^{-8} to 10^{-5}
schist	medium	10^{-3} to 1	10^{-7} to 10^{-1}
*Values at the higher end of the range occur where there is secondary porosity or permeability.			

Appendix G: Porosities and hydraulic conductivities for various rocks and sediments

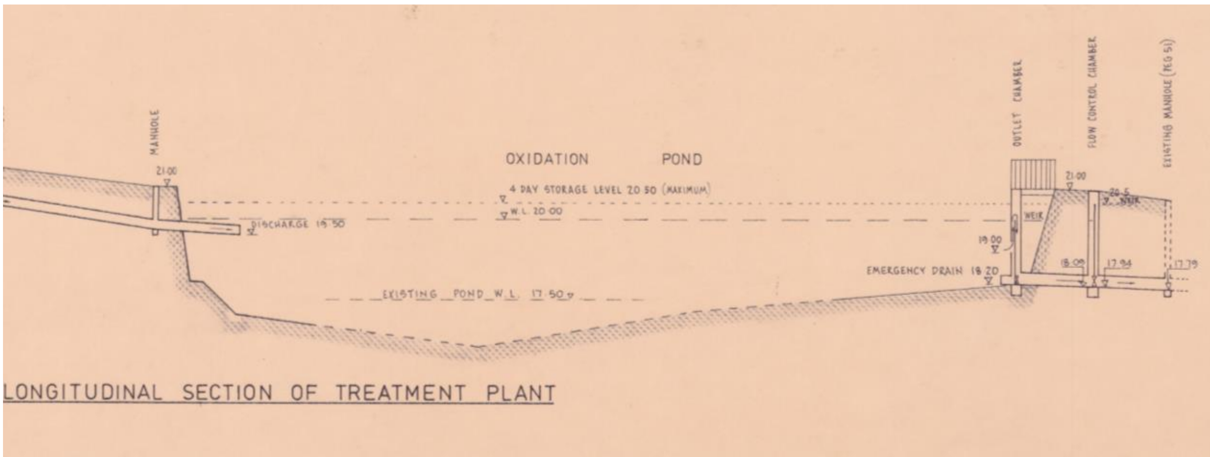
Appendix H



Appendix H: Wairoa Wastewater treatment plant design plans



Appendix H: Wairoa Wastewater treatment plant design plans



Appendix H: Wairoa Wastewater treatment plant design plans

Appendix I

EFFECT OF RECIRCULATION ON NITRIFICATION IN ROCK TRICKLING FILTERS AT SALFORD, ENGLAND

BOD_5 load lb/1000 cu ft/day ($kg/m^2/day$)	Influent BOD_5, mg/l	Influent NH_4^+-N mg/l	Effluent NH_4^+-N, mg/l		Percent nitrification	
			without recirculation	with recirculation	without recirculation	with recirculation
22.6 (0.36)	266	33.9	19.7	13.6	42	60
16.3 (0.26)	235	31.3	16.9	11.8	46	62
11.8 (0.19)	191	32.0	9.7	4.8	70	85
9.2 (0.15)	239	43.9	12.5(1)	2.2	72	95
7.7 (0.12)	165	40.5	11.4	4.9	72	88
5.9 (0.095)	192	40.7	5.7	2.8	86	93
4.6 (0.074)	199	38.3	2.8	0.9	93	98
3.2 (0.051)	206	36.6	0.7	0.4	93	99

[a]Media was blast slag, 8 ft (2.4 m) deep. With recirculation a 1:1 ratio was employed.

(1) original table had 125 mg/L; this was assumed to be a typo and changed to 12.5.

Appendix I: Effect of recirculation on nitrification in rock trickling filters at Salford, England (https://nepis.epa.gov/Exe/ZyPDF.cgi/00000KZ3.PDF?Dockey=00000KZ3.PDF)

* Contact time at existing flow of 0.39 m^3/s (8.8 mgd) plus 33% return rate is 39 minutes.

TABLE 4-3. DESIGN DATA FOR FULL-SCALE TF/SC FACILITIES (FROM MATASCI, et. al., 1988)

Monthly average performance at operating TF/SC facilities.

Parameter	Tolleson April 1983–March 1984			Oconto Falls April 1983–March 1984			Corvallis April 1983–March 1984			Medford April 1984–July 1984		
	High	Low	Average	High	Low	Average	High	Low	Average	High	Low	Average
Influent flow												
Average m^3/s	0.29	0.22	0.27	0.020	0.012	0.016	0 78	0.25	0 46	0.43	0.36	0.39
(mgd)	(6.7)	(5.0)	(6 1)	(0.46)	(0.28)	(0.36)	(17.9)	(5.6)	(10.5)	(9.9)	(8.2)	(8.9)
Influent characteristics												
BOD, mg/L	350	222	277	179	119	146	188	48	108	173	142	157
SS, mg/L	300	192	224	151	100	118	191	112	154	159	119	138
Temperature, °C	—	—	—	19	8	13	22	13	17	22	16	19
Primary effluent												
BOD, mg/L	373	107	173	—	—	—	114	35	70	90	76	81
SS, mg/L	400	57	121	—	—	—	82	56	66	38	29	34
TF effluent												
BOD, mg/L	42.5*	10 4*	22.8*	—	—	—	39	22	30	81	51	66
SS, mg/L	45.9*	9.9*	23.6*	—	—	—	72	54	59	89	39	71
Return sludge SS, g/L	—	—	—	—	—	—	17.2	5.4	11.3	—	—	—
MLSS, mg/L	1620	551	1040	—	—	—	4980	1560	3130	1870	1480	1620
Secondary effluent												
BOD, mg/L	15	4	7	32	14	21	9	5	7	23	14	19
Carbonaceous BOD, mg/L	—	—	—	—	—	—	7	4	5	11	6	8
SS, mg/L	20	4	9	23	6	13	13	7	9	9	6	8

* Intermediate clarifier effluent

TABLE 4-4. PERFORMANCE DATA FROM FOUR FULL-SCALE TF/SC FACILITIES. (FROM MATASCI, et. al., 1988)

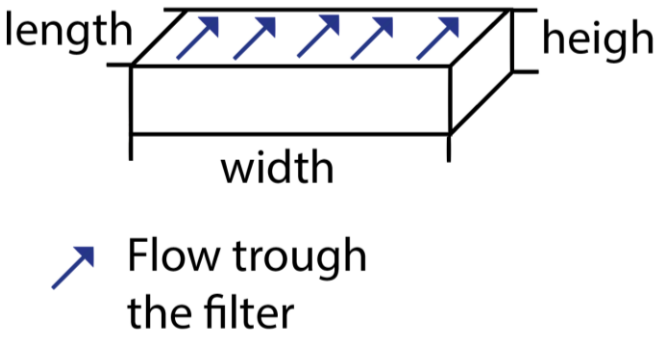
Appendix I: Effect of recirculation on nitrification in rock trickling filters at Salford, England (https://nepis.epa.gov/Exe/ZyPDF.cgi/00000KZ3.PDF?Dockey=00000KZ3.PDF)

TABLE 6-1. SUMMARY OF PERFOMRANCE DATA FOR SELECTED TRICKLING FILTER PLANTS

Facility: Description:	Palm Springs, California Single Stage, Slag Media	Amherst, Ohio Two Plastic Media Filters in Series (No Inter. Clar.) (Single Stage)		Chemung County, New York Two Rock Filters in Series (No intermediate Clarifier) (Single Stage)	Ashland, Ohio Single stage, Plastic media filters (with Solids Contact)	
Influent	T >17°C	T >17°C	T <16°C	T <16°C	T >17°C	T <16°C
BOD (mg/L)	101	62.0	67.1	55.6	104.	85.7
NH_3-N (mg/L)	20	14.7	13.0	11.3	15.5	13.0
Temperature (°C)	23-28	17-20	8-15	11-16	17-22	13.16
Flow (mgd)	7.73	1.86	2.12	5.82	2.7	3.3
Permit[a]						
NH_3-N	None	3	6	None	2	11
BOD (mg/L)	30	10	10	25	10	10
Effluent						
NH_3-N (mg/L)	2.61	1.7	3.0	5.4	3.5	3.6
BOD (mg/L)	7.43	7.5	7.6	10.8	6.	5.8
BOD Loading						
(lbs BOD/1,000 ft^3-d)	11.11	7.3	9.0	15.69	7.6	8.6
(lbs/BOD/1,000 ft^2-d)[b]	0.55	0.24	0.3	0.78	0.253	0.285
NH_3-N Loading						
(lbs NH_3-N/1,000 ft^3-d)	2.15	1.8	1.8	3.2	1.14	1.17
(lbs NH_3-N/1,000 ft^2-d)[b]	0.107	0.057	0.058	0.16	0.038	0.039
Hydraulic Loading						
(gpd/ft^2)[c]	125.	517	589	407	267.	327

Appendix I: Effect of recirculation on nitrification in rock trickling filters at Salford, England (https://nepis.epa.gov/Exe/ZyPDF.cgi/00000KZ3.PDF?Dockey=00000KZ3.PDF)

Appendix J



Appendix J: Clarification of filter measurements

Statement of Authorship

I hereby declare that I am the sole author of this bachelor thesis,

"*Technical feasibility of constructed wetlands to provide for improved environmental and cultural outcomes at Wairoa's wastewater treatment plant*",

and that I have not used any sources other than those listed in the bibliography and identified as references. I further declare that I have not submitted this thesis at any other institution in order to obtain a degree.

Wairoa, 25.7.18 [redacted]

(Place, Date) (Signature)