Kaipara District Council

Mangawhai Wastewater Treatment Plant – Capacity Assessment

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Executive Summary

Currently, the Mangawhai WWTP is reported to be close to its design capacity. Kaipara District Council (KDC) have identified areas of potential growth that could bring the number of connections to a maximum of approximately 7000, from the current 1991 therefore, they would like to check how many connections the existing disposal field and plant can accommodate without any major upgrade work to the plant.

This report is intended to provide an evaluation of the current Mangawhai disposal system, the limitations and the options for increasing disposal into the future as well as the WWTP process and hydraulic capacities. Prior to assessing the capacities of the WWTP, work to identify whether there are any operational improvements that can be made to increase the capacity of the plant was undertaken. Once the maximum capacity is determined, specific bottlenecks within the system can be-identified.

This investigation will provide KDC with the knowledge of the Mangawhai WWTP and disposal field capacity and overall potential so both can be maximised in terms of connections, as well as, be used as a basis to develop a strategy for revision of the consent for the WWTP, and its discharge to the environment if found to be necessary.

Mangawhai Wastewater Treatment Plant

Some operational improvements are recommended to improve the capacity of the Mangawhai wastewater treatment plant. These are:

- During normal operation, it is proposed to increase the total cycle length of the CASS system to 6 hours. During high flow, such as during big rain events the total cycle time can be shortened to a 4 or 3 hour cycle to suit the hydraulic conditions. This proposed aeration is applied for both single and parallel basin operation modes.
- To reduce risk associated with short-circuiting and or an increase of contaminants in the effluent caused by continuous recirculation of the RAS, it is recommended to use the RAS pumps to recycle the MLSS from the main CASS reactor into the first cell during the react stage only. The future recycle rate should be sized to 3 to 7 times the inflow rate entering the Mangawhai Plant (Other similar plants use up to 10 times but this ratio is dependent upon effluent total nitrogen requirements and available carbon).
- Cell 1 and cell 2 within the CASS basins are proposed to be used as designated anoxic zones (Cell 1 always as an anoxic selector and Cell 2 as an anoxic/aerobic swing zone in anoxic mode whenever load conditions permit). This will create an appropriate environment for further removal of nitrogen through the denitrification process. If further removal of ammonia was found to be required, cell 2 can be used as a swing zone allowing aerobic conditions to occur as well.
- For further removal of nitrogen and phosphorus, the supply of oxygen in the main reactor of the CASS system during the react stage can be alternated. This arrangement can be valuable to enhance the removal of nitrogen in times where the loads into the plant are high, such as during Christmas / New Year.

To assess the load carrying capacity of the Mangawhai WWTP, Christmas / New Year influent concentrations were used, as this period represents the highest loads entering the plant. To reach

the maximum capacity, the flow entering the plant was raised until the effluent requirement of the current consent limits were met. This was done while keeping in mind the expected removal capabilities of the tertiary treatment. While following this methodology, the maximum inflow rate to represent the maximum load carrying capacity of the Mangawhai WWTP was found to be 1400 m^3/d. Based on the flow rate of 1,400 m^3/d and the sewage production rate per connection of 650 l/connection/day **the estimated number of connections that can be introduced into the existing Mangawhai WWTP is 2,153. This number of connections is 162 connections higher (8%) than the estimated current number of connections of 1,991.**

To achieve additional loading into the Mangawhai wastewater treatment plant, the following process items would likely require upgrading:

- Inlet Screen – Likely duplication with a Duty / Assist arrangement
- RAS pumps – Increase recycle ratio capability
- Aeration System – At least a new blower to allow dedicated pairs to be assigned to reactors. The As-Built diffuser set up needs to be studied to determine what air flow can realistically and safely be delivered and whether additional diffusers can be fitted in. The actual blower capacities need to be reviewed to determine what spare capacity, if any, these have. To a certain extent, these factors will dictate how much of the time that Cell 2 can be used in an anoxic mode.

Mangawhai Wastewater Disposal Field

During the disposal field site visit evidence of surface water running over the track that separates zone 16A from the unmarked non irrigation area alongside the pond was found. This area had become waterlogged, and the animals that were in the paddock were causing some pugging damage. This observation suggests that the existing disposal fields may be overloaded. However, there is an additional 9 ha of irrigation land that is intended to be used in the near future as an expansion to the current disposal field of 30 ha. This area was inspected and consists of land similar in slope and aspect to that already under irrigation. It was deemed that there was no immediate reason, allowing for setbacks around ponds, gully's, boundaries, tracks and water courses, that this area cannot be used in a similar manner to the existing irrigation block.

There is also an additional 60 ha of land that is yet to be developed for irrigation. This area has not been investigated, however a desk based study of provided literature shows this area to be suitable, and thus it is recommended that a detailed soil mapping exercise is carried out on this land to provide confidence that it is suitable to accept irrigated waste water. This detailed report should identify:

- Slope
- Aspect
- Soil type
- Limiting factors in the landscape
- Current crop and state

- Drainage characteristics, potential for runoff, and thus irrigation capacity

Kaipra DC own yet another additional forestry land with an undulating terrain. The suitability for irrigation will need to be checked before any decision is made on final suitability and expansion. It is also worth noting that when irrigating to forest blocks, there is generally a provision made to take any particular block out for up to 7 years for harvest prep, harvest, clearance, re-planting then establishment of the new crop to a stage where it can be safely irrigated. This needs to be taken into account when planning for any possible irrigation development of this area.

Discussion

Several points for discussion were identified in this report, these are:

- The number of connections the Mangawhai Wastewater Treatment Plant serves is limited by the loads during Christmas and New Year. The Christmas and New Year period only covers about 3 weeks of the year. The Mangawhai Wastewater Treatment Plant capacity can be larger if it was operating to treat the annual average loading rates. Options to mitigate the impact of the Christmas/ New Year high-loading period should be discussed so further treatment capacity can be accommodated. For example, is it feasible to sacrifice the nitrogen removal functionality during this period and still comply with annual compliance limits.

- This report is based on a monthly grab sample influent analysis which does not provide a good representative indication of what is entering the plant. Furthermore, due to absent sampling, no indication of the current CASS system performance and characterisation of the return liquors from the wash water and dewater systems were established. To be able to establish a more accurate performance capacity review of the Mangawhai Wastewater Treatment Plant, an improved monitoring regime should be put in place. The results of this improved monitoring regime can also be used as a basis for any additional upgrade work if it was desired.

- There may be some case to be made for splitting the 'point of compliance' between the treatment plant and the outlet from the storage lagoon to potentially accommodate additional connections. However, more operational data would be required to establish a basis for this.

- Literature review suggests that 60 ha of land is suitable for the discharge of treated effluent. A detailed soil mapping exercise should be carried out on this land to provide confidence that this land is suitable to accept irrigated wastewater as well as assist in the determination of the different feasible disposal method that can be used.

1 Introduction

1.1 Background Information

The Mangawhai Wastewater Treatment Plant (WWTP) provides treatment of wastewater from the Mangawhai settlement sewage scheme. The sewage scheme is comprised of a residential community that feeds into the plant from a conventional gravity system and low-pressure system. The current plant was commissioned during 2008 and operated by Trility. The plant has a reported design capacity of 510 m^3/d during the low season period with a maximum daily inflow rate of up to 5,500 m^3/d during peak wet weather conditions in the peak season.

Currently, the Mangawhai WWTP and disposal field are reported to be close to its design capacity. Kaipara District Council (KDC) have identified areas of potential growth that could bring the number of connections to a maximum of approximately 7000[1], therefore, they would like to check how many connections the existing scheme can accommodate without any major upgrade work to the wastewater treatment plant.

1.2 Purpose of this Report

This report is intended to provide,

- an evaluation of the process and hydraulic capacities of the Mangawhai WWTP while identifying whether there are any operational improvements that can be made to expand the capacity of the plant. Once the maximum capacity is determined, specific bottlenecks within the system will be identified for remedy.
- an overview of the current disposal system, its limitations and the options for increasing disposal into the future.

This overall investigation will provide KDC with the knowledge of the Mangawhai WWTP and disposal field capacity and their potential so it can be maximised in terms of connections, as well as, be used as a basis to develop a strategy for revision of the consent for the WWTP, and its discharge to the environment if found to be necessary.

[1] Mangawhai Community Wastewater Scheme, Capacity and Upgrade Study, Final Report, Trility, January 2016

1.3 Structure of this Report

This report is divided into sections as summarised in Table 1.

Table 1: Summary Structure of this Report.

Section	Key Items Covered
1. Introduction (presented above)	Provides background information and the scope of this report
2. Process Unit Elements Hydraulic Capacity.	Provides an overview of the current plant major process elements as well as the elements hydraulic capabilities.
3. Basis of Investigation	Provides the key design parameters, such as influent flow, influent loads, diurnal curves and discharge conditions upon which the capacity of the Mangawhai treatment plant work was established.
4. Plant Treatment Capacity Determination	Provides background on the existing plant operation, proposed details of an optimise operation and the capacity investigation data and findings.
5. Disposal System	Provides an overview of the current disposal system, the limitations and the options for increasing disposal into the future.
6. Conclusion and Discussion	Provides the conclusion of the disposal field, and process and hydraulic capacity investigations as well as notes for discussion to be used as a basis for any further future action / work.

2 Process Unit Elements Hydraulic Capacity

2.1 Background

This section presents an overview of the existing plant major process elements along with details regarding the hydraulic elements and design capacity. The information used in this section is based on previous studies completed by Trility[2], the original design specification and information gathered by Opus through site visits and communication with suppliers and the treatment plant operators.

The hydraulic and design capacities of the process elements presented in this section will be used and compared with the finding of the process capacity study presented in Section 4 to evaluate what are the limiting process unit elements that require an upgrade. The conclusion of this comparison will be presented in Section 6.

[2] Mangawhai Community Wastewater Scheme, Capacity and Upgrade Study, Final Report, Trility, January 2016

2.2 Current Plant Configuration

The existing Mangawhai WWTP is based on the concept of biological treatment of screened wastewater and a tertiary treatment. The plant's main elements are as follows:

- Screening and associated screenings wash press
- Flow splitter chamber
- Cyclic Activated Sludge System (CASS) with Return Activated Sludge (RAS) to bioreactors, and Waste Activated Sludge (WAS) to the sludge management system.
- Tertiary treatment that includes filters, and disinfection using chlorine and / or UV systems
- Sludge management system that includes an aerobic digester tank and a belt filter press system.
- Odour control system

A schematic of the Mangawhai WWTP is shown in Figure 1, while an overview and the estimated capacity of each element is provided further below.

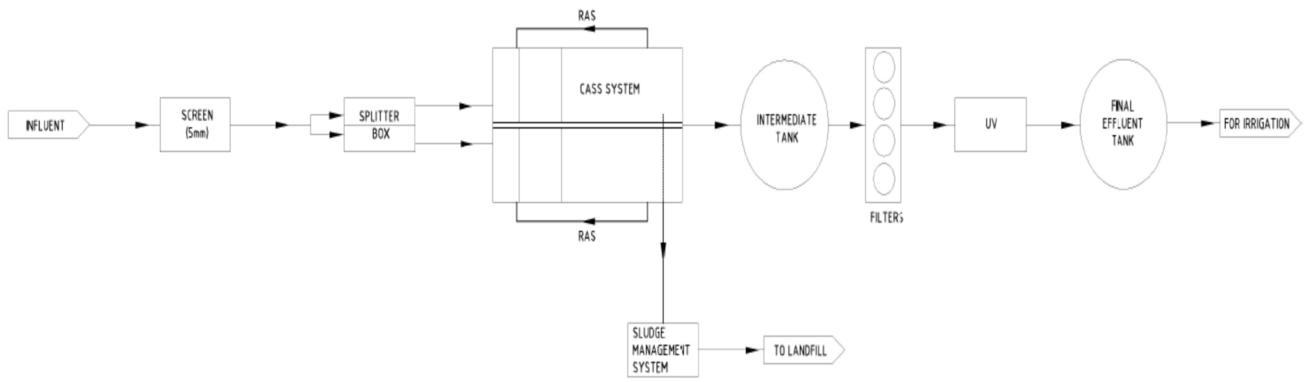

Figure 1: Basic Schematic Diagram of the Mangawhai WWTP.

2.2.1 Inlet Works (Screening)

Influent (raw sewage) is pumped directly to the WWTP from a pump station, which receives raw sewage from the Mangawhai wastewater reticulation system. The inlet works (Figure 2) is an integrated single SFC-T5 5 mm screen press and compactor, capable of passing up to 150 l/s at an upstream pressure of 0.525-meter head[3]. Although it seems that the current configuration can allow this flow to occur, the flow rate of 150 l/s represents a clean water condition. Therefore, it is uncertain how the screen will behave at this flow rate with Mangawhai raw influent sewage. On this basis, for this report, the design capacity of 70 l/s was used[4]. This design capacity of 70 l/s was designed to match the maximum pumping capacity of the terminal pump station.

[3] J. Gill (Smith & Loveless New Zealand), communication and reference, 22 November 2016
[4] Mangawhai Wastewater Treatment Plant Design Report, Water Infrastructure Group, April 2009.

Figure 2: Mangawhai WWTP Inlet Screen.

2.2.2 Splitter box

Flow from the screen is directed to a Splitter Box with internal weirs and an overflow arrangement. The Splitter Box allows distribution of the flow from the screen to the biological reactors. During low demand, either side of the Splitter Box can be closed manually by an isolation valve. This will ensure only one CASS Basin is in operation.

Figure 3: Splitter box internal view

The hydraulic review completed by Trility suggests that a flow of up to 80 l/s can be used for each of the splitter box cells, therefore the expected maximum total flow that the splitter box arrangement can accommodate is 160 l/s. Any flow higher than 160 l/s will be directed into the Return Liquors Pump Station using the overflow weir within the Splitter Box.

2.2.3 Secondary Treatment – CASS System

From the Splitter Box, the screened wastewater continues to the secondary treatment reactors within the CASS system. The CASS reactor basin is divided by baffle walls into three sections (Zone 1: Selector, Zone 2: Secondary Aeration, Zone 3: Main Aeration), refer to Figure 4. The total effective volume of each of the CASS basins is reported to be approximately 870 m^3.

Figure 4: CASS System

The CASS system is a combination of a biological selector and sequence process reactor. A repeated sequence of aeration and non-aeration can be used to provide aerobic, anoxic and anaerobic process conditions, which in combination with the aeration intensity, favour nitrification, denitrification and biological phosphorus removal. The CASS system is essentially a version of the generic IDEA (Intermittently decanted, extended aeration) process configuration. This is a batch style process that employs continuous feed to all operational reactors whether they are in 'React', 'Settle' or 'Decant' phases of their operational cycle.

Alternatively, as is the case with most IDEA plants, the front sections of the reactor can be operated continuously without air to form permanent anoxic zones and enhance denitrification. However, the Mangawhai plant has not been operated in this mode.

The aeration system comprises of a set of 3 blowers. Each is capable delivering a flow rate of 470 m^3/hr[5]. During peak season, all three blowers can operate together delivering a flow rate of 1,360 m^3/hr. It is normal for IDEA plants to operate on a 6 hour cycle (4 hrs react, 1 hr settle, 1 hr decant), stepping down to 4 hours in wet weather and 3 hours in extreme wet weather. Mangawhai

[5] Mangawhai Wastewater Treatment Plant Design Report, Water Infrastructure Group, April 2009.

has always been operated on a 4 hour dry weather cycle. This has allowed the 3 blowers to be shared between the two reactors without any aeration timing overlap.

Each CASS reactor has its own decanter arrangement. The decanter is used to transfer the treated effluent from the CASS system to an intermediate storage tank. Each decanter has 4 outlets of 300mm nominal diameter. Flow through the decanter is dictated by how far under the water the front edge of the decanter trough is, and the intermediate tank water level. Based on the CASS minimum operating water level of 17.480 mRL and the intermediate tank maximum operating level of 16.300 mRL, the design peak flow to flow through each decanter system is estimated to be 200 l/s

2.2.4 Return and Waste Activated Sludge

To maintain a constant population of micro-organisms in the reactor, activated sludge (RAS) is returned continuously from the end (decanter side) of the CASS system back into the reactor Zone 1. The waste activated sludge (WAS) from the end (decanter side) of the CASS system is directed to the sludge treatment system, which is used to stabilise the solids and facilitate separation of the liquid from the solids.

Figure 5 shows the RAS and WAS pumps.

Figure 5: RAS and WAS pumps

The RAS and WAS pumps are Grundfos SEV.80.80.11.4.50B. The RAS or WAS generated flows will be determined by the water level of the CASS system, the viscosity of the liquid, the head losses in the lines and the discharge point water levels (CASS cell 1 water level in terms of the RAS pump and the sludge storage tank water level in terms of the WAS pump).

Table 2 provides the expected capacities of the WAS and RAS pumps during the different operation scenarios based on Trility Hydraulic investigation. The RAS rate capability appears lower than would normally be expected in this type of plant.

Table 2: Estimated capacity of the RAS and WAS pumps

Description	Estimated Capacity
RAS Pump	8.5 – 10 l/s
WAS pump	9.5 – 18 l/s

Notes:

A The capacities are based on 50Hz operation curve

B Maximum RAS pump capacity is based on one single pump in operation.

C Maximum WAS pump capacity is based on two pumps operating in parallel.

2.2.5 Intermediate Storage Tank

Treated effluent from the CASS decanter system is directed to the intermediate storage tank (Figure 6), with an approximate effective volume of 475 m^3 (estimated effective volume based on the As-Built drawings), before continuing to the tertiary treatment system. This tank provides a buffering facility between the very high rates of decant flow and the desire to minimise flow rate through the tertiary processes and minimise their size and cost and maximise their usage.

Figure 6: Intermediate Storage Tank located downstream the CASS System

Two submersible Grundfos SE1.80.100.75.4.50B pumps are situated in the Intermediate Storage Tank responsible delivering the secondary treated effluent from the Intermediate Storage Tank via the tertiary treatment to the Final Effluent Tank. The Intermediate Transfer Pump/s generated flow is determined by the water levels of the Intermediate Storage and Final Effluent Tanks and the different system curve scenario.

Table 3 provides the expected capacities of the Intermediate Transfer Pump/s during the different operation scenarios based on Trility Hydraulic investigation.

Table 3: Estimated capacity of the Intermediate Transfer Pump/s

Description	Estimated Capacity
Intermediate Transfer Pump	32 – 48 l/s

Notes:

A The capacities are based on 50Hz operation curve

B Maximum pump capacity is based on two pumps operating in parallel.

2.2.6 Tertiary Treatment

The tertiary treatment elements receive flow from the Intermediate Storage Tank through a pair of feeding pumps. The tertiary treatment includes a GEBAL AQUASAFE filtration system and two ultraviolet (UV) disinfection reactors.

2.2.6.1 Filtration system

The GEBAL AQUASAFE filtration system (Figure 7) is the first element within the Tertiary Treatment that is designed to improve UV disinfection ability. This filtration system contains four deep bed multimedia fiberglass reinforced plastic filter vessels. The system is fully assembled with stainless steel pipe work, compressor, pressure sensors, flow meter, valves, and a control panel.

The GEBAL AQUASAFE filtration system contains a media called Activated Filter Media (AFM) manufactured by Dryden Aqua. The AFM media is a glass filter media which claims to provide properties of absorption and catalysis, as well as, acting as a physical filtration mechanism.

Figure 7: Filtration system

The flow meter determines the number of filters that need to be operating. The system specification indicates flow rate range from 7 l/s (one vessel in operation) to 26 l/s[6] (four vessels in operation).

Backwash is required to ensure good operation of the filtration system. The system uses final effluent from the final effluent tank to individually backwash each vessel in a predetermined sequence, which is programmed into the PLC. Actual backwash flow of 24 m^3 per a wash cycle was reported by Trility to be used with only one wash cycle per a day during normal operation. The frequency of the cleaning cycle of the filter system is determined by the trans-filter pressure, however, Trility experience suggests backwash water volume would be approximately 10% of the flow entering the plant[7].

The backwash is then directed to the Return Liquors Wet Well so it can be returned to the head of the plant.

Communication with the supplier, GEBEL, have advised that the company no longer fabricate this filtration system and no data regarding the performance of the system could be obtained.

[6] Taken from the supplier product manual
[7] R. Johnson, personal communication (email), 8 December 2016.

2.2.6.2 UV system

Downstream of the filter is the UV system (Figure 8). The UV system is a reactor arrangement, consisting of two sets of reactors operating in series. The UV system appears to be Berson IL200+WW with 2 lamps in each reactor. The supplier of Berson in New Zealand is Davey, however communication with Davey revealed that they have very limited information on the system installed in Mangawhai WWTP.

Figure 8: UV system

The system hydraulic flow limit is estimated to be 67 l/s[8], however it is not clear what disinfection performance is expected at this flow rate. Based on this, the design UV flow requirement of 26 l/s was used as the maximum capacity of the UV system[9]. Currently, the UV system seems to have mechanical issues and is not in operation.

Disinfection prior to the Final Effluent Tank is undertaken using Sodium Hypochlorite. The Sodium Hypochlorite dose is continuous and proportional to the flow entering the tank.

[8] Based on data provided from Davey New Zealand for the InLine 200 + WW summit UV system.
[9] Mangawhai Wastewater Treatment Plant Design Report, Water Infrastructure Group, April 2009.

2.2.7 Final effluent Tank

The filtered and disinfected effluent is directed to the final effluent tank (Figure 9) with an approximate effective volume of 470 m^3 (estimated effective volume based on the As-Built drawings) for storage before it is pumped to the irrigation field.

Figure 9: Final Effluent Storage Tank

2.2.8 Sludge Treatment

The sludge treatment at Mangawhai WWTP consists of 4 unit processes, the waste sludge holding tank (WAS Tank), polymer system, gravity drainage deck and belt press filter.

The sludge generated in the CASS system is transferred into the 180 m^3 (estimated effective volume based on the As-Built drawings) sludge holding tank (Figure 10) before it continues to the belt press system (Figure 11) for dewatering.

Figure 10: Aerobic Digester

Figure 11: Final Effluent Storage Tank

Communication with the Belt Press System supplier, Ecomacchine, confirmed that this system has a maximum hydraulic flow rate of 15 m^3/hr (4.16 l/s) and maximum dry solids flow rate of 125 kg/hr. The optimum dry solids feed is in the order of 60 to 80 kg/hr in which the expected performance claimed by the supplier is 20 to 22% dry solids. However, current operation suggests dry solids concentration of around 15%.

Currently, the belt press system does not operate daily, normally the holding tank will keep the sludge aerated until the dewatering process is selected. No evidence as to the amount of extracted water was provided, however, communication with Trility suggests that for every 50-80 m^3 of WAS, about 30 m^3 of wash water is used, this in addition to about 96% of reject water (based on 17% dry solids). The water out of the belt press system is directed to the Return Liquors Wet-Well where it joins with the filter wash water and is directed into the head of the plant.

2.3 Summary of Unit Processes Hydraulic Capacities

Different process units along the main treatment stream were hydraulically analysed and the available capacity of each was determined. Table 4 presents the summary of the maximum capacity of each process unit.

Table 4: Summary of the maximum flow capacity of the different process units

Description	Estimated Hydraulic Capacity (l/s)
Screen	70
Splitter Box	160
CASS Decanter System	200
RAS pump	8.5
WAS pump	14
Intermediate Transfer Pump	48
Filtration System	26
UV system	26
Belt Press System	4.16

Note:

A The screen and splitter box capacities include the associated pipework of these systems

3 Basis of Investigation

3.1 Introduction

This section provides the key design parameters, such as influent flow, influent loads, diurnal curves and discharge conditions upon which the process and hydraulic capacity of the Mangawhai wastewater treatment have been developed.

KDC would like to assess the maximum capacity of the existing Mangawhai WWTP in terms of a number of connections. A previous report suggests that the current Mangawhai scheme comprises of 1991 properties[10]. Assuming that each property contains only one connection, out of the 1991

[10] Mangawhai Community Wastewater Scheme, Capacity and Upgrade Study, Final Report, Trility, January 2016.

connections, 1,565 connections use a gravity system to deliver the sewage towards the WWTP, while the remaining 426 connections use a low-pressure sewer system. The number of 1991 connections is slightly higher than the 1800 connections mentioned in the MCWWS Community Advisory Panel Report[11]. However, it is understood that the number of connections mentioned in the Trility report includes commercial activities such as the school, shopping center, and restaurants that have been converted into connection equivalent.

During Christmas and New Year the occupancy rate per house is expected to rise considerably, however, the water use and therefore the wastewater discharge rate per person is expected to be reduced due to the nature of the water scheme (rain collection), manner in which the additional people use 'facilities' and low precipitation rate. Currently, there is no source of evidence that shows what this rate will be. For the purpose of this investigation, it is important to define what the term connection means in terms of flow, as this time period represents the highest inflow rate and full utilisation of the households (connections). On this basis and for the purpose of this investigation, the daily flow rate recorded during peak season (Christmas and New Year) was divided by the estimated current connection of 1991 taken from the Trility report to provide a daily inflow rate per connection. Once this rate is known it will be used to evaluate the maximum number of connections that the Mangawhai WWTP can accommodate.

3.2 Wastewater (Influent) Flows

As explained in Section 2.1, it is important to know what the current inflow rate entering the Mangawhai WWTP is so the inflow rate per connection can be evaluated and be used to estimate the maximum number of connections that can be served by the current WWTP. This section presents the summary of the historical flow monitoring taken at the inlet of Mangawhai WWTP, as well as the recommended flow rates and peak factors to be used for the purpose of the process and hydraulic investigations undertaken in this report.

3.2.1 Existing Influent Flow

Table 5 presents the summary of the average flow data collected during March 2012 to January 2016. The flow rates are divided into four groups, all year average, Autumn/Winter average, Spring/Summer average and Christmas/New Year average, so comparison over the different times of the year can be observed easily. The concept behind this breakdown is due to the fact that Mangawhai is a holiday destination in which the number of residents tend to change during the different times of the year and hence the influent flow is expected to fluctuate.

[11] MCWWS Community Advisory Panel Final Report, Kaipara District Council, July 2015

Table 5: Average flow data collected during March 2012 to January 2016.

Year	All Year Average Daily Flow (m^3/d)	Autumn/Winter Average Daily Flow (m^3/d)	Spring/Summer Average Daily Flow (m^3/d)	Christmas/New Year Average Daily Flow (m^3/d)
2012	301	306	275	598
2013	293	262	293	664
2014	332	397	329	681
2015	354	318	349	701
Total Average	**327**	**292**	**323**	**682**

Notes:

A autumn / winter period includes sampling taken from March to August

B spring / summer period includes sampling taken from September to 23 of December and mid-January to the end of February

C Christmas / New Year period includes sampling taken from 24 of December to mid-January the following year.

The annual average inflow rate presented in Table 5 of 327 m^3/d is considerably lower than the original off-peak design flow (ADWF) of 510 m^3/d, as suggested in the "Project Deed" schedule within the Design Report[12]. This comparison indicates that over the entire year the wastewater treatment plant is not being used to its full design capacity.

Table 5 shows that there is a slight increase in inflow entering the Mangawhai WWTP during the spring and summer period over the autumn and winter period. The flow rate during these times of the year is very close to the annual daily average. However, the average flow rate during Christmas and New Year is in the order of double the annual average.

Based on what was explained in Section 4.1, to calculate the critical flow rate produced from a "connection" the maximum recorded daily flow rate during Christmas and New Year was divided by the estimated number of connections (1,991) (Refer to Section 3.2). Table 6 presents this calculation based on the highest daily flow of 1300 m^3/d recorded during 15th of December 2014.

Table 6: Calculated discharge rate per connection based on peak day monitored during 15th of December 2014

Estimated number of connection	Peak Dry Weather Flow at 1991 connections recorded during 15 December 2014 (m^3/d)	Estimated Flow per a Connection during Peak season (l/connection/d)
1991	1300	653

The flow rate of 653 l/connection/day is within the 630 to 700 l/connection/day range used in the Trility Report[13]. **On this basis, to evaluate the maximum number of connections that the Mangawhai WWTP can treat, a flow rate of 650 l/connection/day was used.** This is assuming that any new connections in the future will be of a similar nature to the existing connections.

[12] Mangawhai Wastewater Treatment Plant Design Report, Water Infrastructure Group, April 2009.

[13] Mangawhai Community Wastewater Scheme, Capacity and Upgrade Study, Final Report, Trility, January 2016.

3.2.2 Flow Rate Variations and Peak Wet Weather Flow Determination

The peak wet weather flow (PWWF) is normally accommodated in the design as a peaking factor that is applied to the average dry weather flow (ADWF[14]). However, the ADWF was difficult to evaluate correctly from the Mangawhai monitoring data available, as it is directly linked to the precipitation rate. Therefore, the average inflow rates presented in Table 5 were used. Based on this, the peaking factor is calculated by dividing the maximum flow rate by the average flow.

The calculated annual and Christmas New Year Peak Factors are presented in Table 7.

Table 7: Wet Weather Peaking Factor (PWWF:ADF) calculation for the Mangawhai WWTP

Year	Average Annual Daily Flow (m^3/d)	Maximum Annual Daily Flow (m^3/d)	Christmas/ New Year Period Average Daily Flow (m3/d)	Christmas/ New Year Period Maximum Daily Flow (m^3/d)	Calculated Annual Peak Factor	Calculated Christmas/ New Year Period Peak Factor
2012	301	894	598	925	3.0	1.5
2013	296	971	664	971	3.3	1.5
2014	332	1300	681	1024	3.9	1.5
2015	354	1024	701	1142	2.9	1.6
Total Average					3.4	1.5 (3)

Notes:

A The average annual and Christmas / New Year daily flows were taken from Table 5.

B Peak Factor of 3 (shown in brackets) represent of peak factor of the maximum flow recorded during Christmas and New Year over the annual average flow.

In New Zealand, PWWF: AWF peaking factors of between 2 and 5 are not uncommon, and are used for determining the flow to full treatment (FFT), which is the effective design capacity of the wastewater treatment plant under maximum flow conditions. The original design[15] indicates higher peaking factor than the one observed and shown in Table 7, these are:

- Peak Wet Weather Flow Factor over off peak season of 4.
- Peak Wet Weather Flow Factor over peak season of 5.

However, the design peak factors mentioned above are applied to the annual average dry weather flow and not on the annual average flow as suggested in Table 7. Although the current annual dry weather flow entering the Mangawhai WWTP is unknown, the peak factor of 4 over off peak season is very similar to what was found and presented in Table 7. However, a peak factor of 5 to represent the peak season is relatively excessive, not very typical in New Zealand and does not match the historical data as presented in Table 7.

[14] ADWF is normally defined as the average flow calculated from days that have had 7 or more preceding days without rain.

[15] Mangawhai Wastewater Treatment Plant Design Report, Water Infrastructure Group, April 2009.

Table 7 shows that the peak factor over Christmas and New Year is less than the peak factor calculated over the entire year.

Based on the above and considering that any further development in the Mangawhai will continue on the same basis as the current water supply and collection system schemes, **the recommended peak wet weather flow factor *for the purposes of this report* is 3.5 (applied on the design ADF).** This peak factor covers both off-peak and peak season periods.

3.2.3 Peak Instantaneous Flow

In addition to the FFT, peak instantaneous flow is required to determine the capacity of the hydraulic elements within the wastewater treatment plant (for a relatively small WWTP that attracts tourism during some periods of the year, such as the Mangawhai WWTP, the hydraulic capacity of the plant will be greatly influenced by both diurnal daily peaks, as well as peak seasonal flows).

To determine the peak instantaneous flow, continuous monitoring (every 15 minutes) of the inflow data entering the Mangawhai WWTP during 2015 was used and analysed. To provide an indication of the instantaneous peak, two representative days, 2nd of March 2015 and 28th of December 2015, were selected to characterise the peaks during off season and peak season respectively. These days were selected as their patterns were shown to be relatively unfluctuating and without any odd spikes, as well as, representing a dry weather scenario to avoid any additional flow occurring during wet weather conditions, which would already be accounted for when calculating the FFT and therefore to avoid any possible double counting.

Figure 12 and Figure 13 present the diurnal curves used to calculate the peak instantaneous flow entering the Mangawhai WWTP during off-peak and peak seasons.

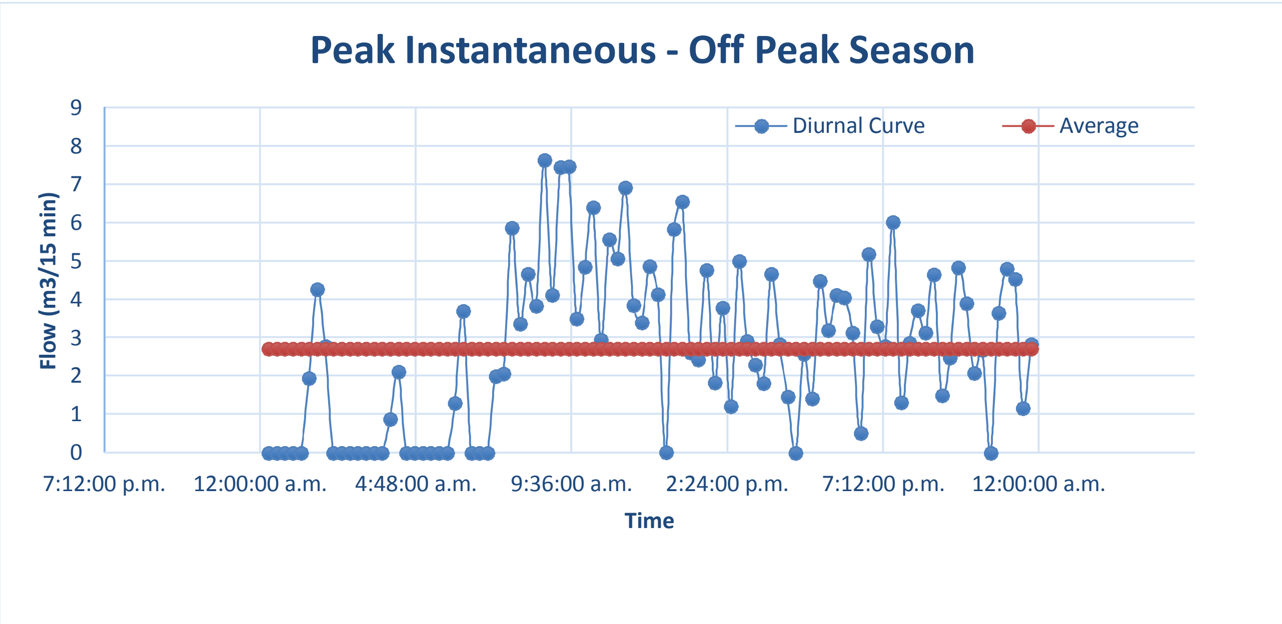

Figure 12: Diurnal Curve monitored during 2nd of March 2015 to represent peak instantaneous flow during off-peak season.

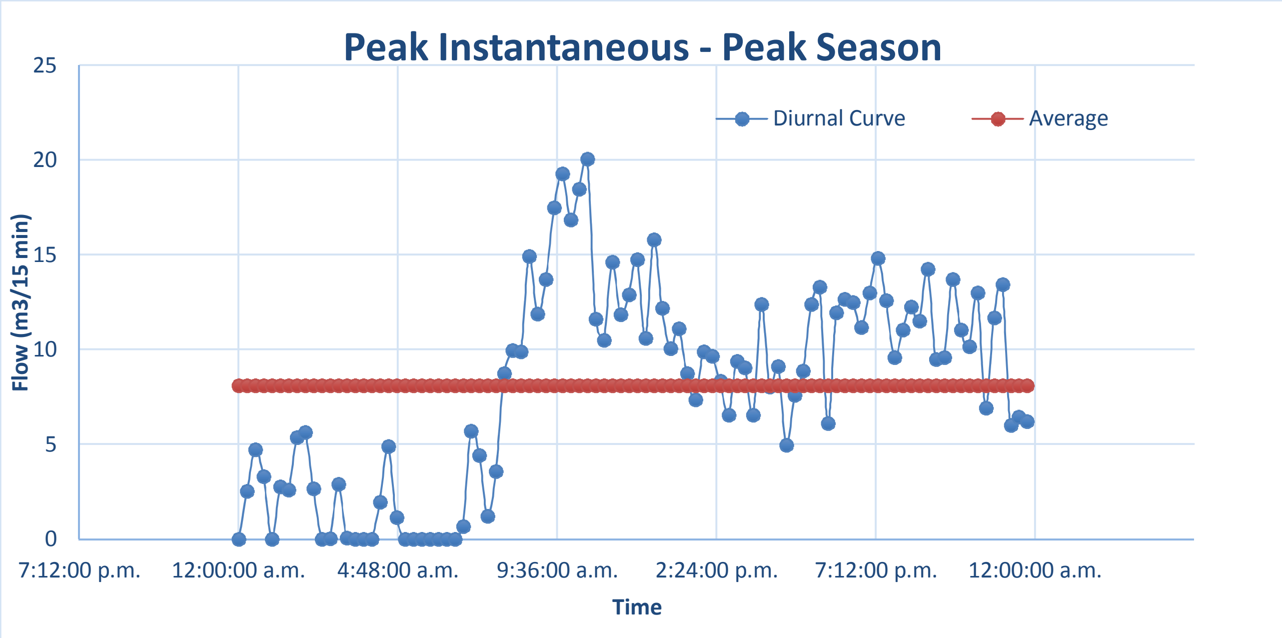

Figure 13: Diurnal Curve monitored during 28th of December 2015 to represent peak instantaneous flow during peak season.

Table 8 shows the instantaneous peak flow factor for both off-peak and peak season, calculated based on the diurnal curves and the average rates shown in Figure 12 and Figure 13.

Table 8: Instantaneous Peak Flow Factor Calculated for both off-peak and peak seasons

Period	Peak Instantaneous Flow Factor
Off-Season	2.8
Peak-Season	2.5

Although the peak instantaneous flow entering the Mangawhai WWTP is currently restricted by the feed pump capacity and will vary in accordance with the operating water level in the pump station sump, this peak should be determined and be used for the purpose of any future capacity increase to determine the plant hydraulic capabilities.

Based on the above and considering that any further development in the Mangawhai scheme will continue on the same basis as the current scheme, the recommended peak instantaneous flow factor for the purposes of the hydraulic investigation is 2.5. ***The instantaneous peak factor of 2.5 is recommended to be applied to the ADF, in addition to the peak seasonal flow discussed in 3.2.2 of 3.5, giving a total peak of 8.75.***

The peak of 8.75 will only be applied to elements prior to the CASS System, such as the screen, the splitter box, and the associated pipe works. Any instantaneous peak after the CASS system will be equalised by the CASS reactors' volume.

3.3 Influent Characteristics

The influent monitoring of the Mangawhai WWTP was completed by Trility using a grab sample method. Table 9 presents the summary of the average values collected during 2013 to 2015. As with the flow data, the influent characteristics are divided into four groups, full year average, Autumn/Winter average, Spring/Summer average and Christmas/New Year average so comparison over the different times of the year can be observed easily.

Table 9: Average Influent Characteristics of Mangawhai WWTP collected during 2013 to 2015.

Year	Season	Average Value (mg/l)						
		TSS	BOD_5	TP	TN	NH_3-N	NO_3-N	COD
2013	Autumn/Winter	505	282	12	84	79	0	795
	Spring/Summer	362	245	12	89	83	0	588
	Christmas/New Year	401	469	15	109	101	0	900
	All year	443	287	12	88	82	0	718
2014	Autumn/Winter	362	415	14	84	73	0	657
	Spring/Summer	333	412	11	62	51	0	333
	Christmas/New Year	460	421	14	128	109	0	939
	All year	364	363	13	82	71	0	564
2015	Autumn/Winter	390	299	12	99	78	0	765
	Spring/Summer	413	336	13	102	82	0	696
	Christmas/New Year	444	417	14	128	110	0	966
	All year	421	311	12	97	78	0	795
Total	Autumn/Winter	419	265	13	89	77	0	739
	Spring/Summer	369	331	12	84	72	0	539
	Christmas/New Year	435	436	14	122	107	0	935
	All year	409	320	12	89	77	0	693

Notes:

A autumn / winter period includes sampling taken from March to August

B spring / summer period includes sampling taken from September to November and February

C Christmas / New Year period includes sampling taken from December and January of the following year.

The suggested concentrations over the Christmas / New Year period shown in Table 9 are higher than the rest of the year, as well as, than the expected typical domestic raw influent characteristics. However, this is not a surprise, as these concentrations represent the most critical time of the year where the expected discharge per person is the lowest due to the nature of habitation at that time of year (staying in tents, fewer showers per person, lesser lighter clothes for laundering) and the low precipitation rate taking place in this time of the year, forcing residents to conserve water.

During 6th, 7th and 9th of February 2016, hourly samples were taken place. The summary of these sampling results is presented in Table 10

Table 10: Hourly Influent sampling results taken during 6, 7, 9/2/2016 from 8:00 a.m. to 5:00 p.m.

Date/time	Average Value (mg/l)						
	TSS	BOD_5	VS	TKN	NH_3-N	TP	COD
6,7,9/02/2016 from 8:00 a.m. to 5:00 p.m.	640	504	577	126	91	N/A	1900

Notes:

A TP and NH3-N values were not available

B VS value was recorded, giving TSS/VS ratio of 1.1

When comparing Table 9 Summer / Spring average values, with the information presented in Table 10, it can be observed that all of the concentrations, except the ammonia level, are higher during the hourly samples taken on the 6th , 7th and 9th of February 2016. However, the samples taken during February 2016 are not representing a full day of results and only cover the working hours, therefore cannot be compared or used.

The sampling results shown in Table 10 are very differences from the results of the grab sample, therefore indicate that the grab sampling could be significantly under estimating the actual concentrations, and therefore loads coming in to the plant. For that reason it is recommended that all future influent sampling is done by composites. Preferably flow weighted composites.

The proposed influent characteristics to be used for the purpose of this investigation is based on the total average during the Christmas and New Year period data, as this period is shown to have higher concentrations. This high concentration alongside the high inflow rate (as shown in Table 5) deliver the highest loads entering the Mangawhai Wastewater Treatment Plant and a representative period of the most critical scenario during the year which the plant should be designed to treat.

Furthermore, the current resource consent requires the treated effluent to be sampled once every seven days using a grab sample method, as well as a flow weighted 24-hour composite sample. On this basis, to ensure consent compliance, the Christmas / New Year results as they are representing the most critical period should be used as a basis for this investigation.

Table 11 presents the influent characteristic used in this investigation

Table 11: Mangawhai influent characteristics used for the capacity study of the Mangawhai WWTP

Period	Concentration (mg/l)							
	TSS	ISS	BOD_5	TP	TKN	NH_3-N	NO_3-N	COD
Christmas / New Year	435	40	436	14	122	107	0	935

Note:

A ISS value is based on the TSS/VS of 1.12 recorded during the February 2016 samples presented in Table 10

3.4 Current Effluent Quality Targets

The current discharge resource consent of the Mangawhai WWTP is due to expire in September 2042. Presented in Table 12 are the concentration limits determined under this consent (AUT.014969.01.02) at the end of the treatment plant process, before being directed into the irrigation field.

Table 12: Mangawhai Resource Consent Discharge Limits Summary

Description	Units	Maximum	Median	Average	90%tile
Discharge quantity	m^3/d	5,500			
E coli	MPN		10		100
Total Dissolved Solids	mg/l			500	
Total Nitrogen	mg/l			30	
Phosphorous	mg/l			15	
Total Suspended Solids	mg/l			10	
cBOD5	mg/l			10	

3.5 Physical Data of the Reactors

The biological system within the Mangawhai WWTP consists of two CASS process basins. Each CASS system includes three cells: selector, secondary aeration and the main reactor. Table 13 presents the sizing of each cell as used in this investigation.

Table 13: Physical sizes of the CASS system

System	Element name	Volume [m3]	Area [m2]	Depth [m]
CASS 1	Selector	48	9.6	5
	Secondary Aeration	131	26.2	5
	Main Reactor	690	138.0	5
	Total Basin	869	173.8	5
CASS 2	Selector	48	9.6	5
	Secondary Aeration	131	26.2	5
	Main Reactor	690	138.0	5
	Total Basin	870	173.8	5

Notes:

A The 5 m water depth is based on top water level (TWL) as shown in the As-Built drawings

B The sizes of the main reactor zone, the secondary aeration zone, and the selector zone were estimated based on the site photos and measurement from the As-Built drawings

It should be noted that the total basin volume shown in Table 13 is larger than the 798.75 m^3 calculated based on the design report[16] and based on total length of 21.3 m, width of 7.5 m and TWL of 5 m. However, for the purpose of this investigation, the reactor measurement of the construction drawings as shown in Table 13 were used.

[16] Mangawhai Wastewater Treatment Plant Design Report, Water Infrastructure Group, April 2009.

4 Plant Treatment Capacity Determination

4.1 Background

To determine the maximum load treating capacity of the Mangawhai treatment plant, firstly a process model was built in accordance with the current plant operation (refer to Section 4.2). Once the model was completed, verification of the model against site information was established. After the verification work was completed, simulation to determine the capacity of the Mangawhai WWTP using Christmas / New Year influent concentration levels (refer to Table 11) was undertaken. While using the Christmas / New Year influent concentrations, the flow entering the plant was raised until the effluent requirement of the current consent limits (refer to Table 12) were met. This was done while taking into consideration the expected removal capabilities of the tertiary treatment. Once the inflow rate was determined and because the influent concentration feed is known, the maximum load carrying capacity of the plant was established.

The model verification work and data, the capacity assessment details and the results are described and shown in this chapter.

4.2 Existing Controls

During the off-peak season (approximately between the middle of February to the beginning of December) the Mangawhai WWTP operates on one CASS basin only, while during the rest of the time, which is considered peak season, two CASS basins will be in operation. Both the one and two basin configurations would normally run on a four-hour cycle, however, during higher inflow rates, the cycle duration is programmed to be reduced to a three-hour cycle[17]. The changeover between the cycles is controlled by the water level within the reactor in the first 30 minutes of the cycle.

Table 14 presents the different stages of the CASS system and the operation time for each stage.

Table 14: Existing operation cycle time

Stage	Normal Condition Stage Operation Time (min)	High inflow Stage Operation Time (min)
Fill	Continues	Continues
React	120	60
Settle	60	45
Decant	60	45
Total Cycle	**240**	**180**

Notes:

A during the operation of two basins, two-hour lag between stages will occur.

Return activated sludge (RAS) is currently continuously directed to the first cell from the main CASS reactor, while the Waste Activated Sludge (WAS) removal is done on a volumetric basis and occurs during the decant stage only, ensuring the maximum possible WAS concentration is wasted.

[17] R. Johnson (Trility),personal (via email) communication, 31 October 2016

When two basins are in operation, the stage cycles are programed to operate with a two hour offset. For example, during normal conditions, when one CASS basin is in react stage, the other CASS system will be either in settle or decant stage. This way the same blowers can be shared between the two basins and the decant hydraulic requirements can be minimised.

Each cell within the CASS systems has the ability to be aerated or not. Table 15 presents the aeration duration, expressed in percentage, during each of the CASS stages.

Table 15: Existing aeration control

PERIOD	Normal and High Inflow Condition	
	Aeration (%) [cell1] [cell2] [cell3]	No. of "Air On" cycles (-)
React	[100] [100][100]	Constant aeration DO control
Settle	no aeration	
Decant	no aeration	

Notes:

A cell 1 = selector; cell 2 = secondary aeration; cell 3 = main reactor

B the aeration control is a function of the dissolved oxygen level set point of 2.5 mg/l.

C cell 1 (selector) is aerated using coarse aeration diffuser while the cells 2&3 using coarse aeration.

4.3 Model Validation

The process model of the existing plant was set up on the BioWin platform. To confirm the model reliability it is normally necessary to verify the model against actual data. The objective of the model validation is to try and obtain an approximate match that would provide insight on the model output accuracy. The model is then used to assess the performance of the plant under different loading or modification scenarios.

Biowin models the biological component of the treatment plant. However, the current sampling regime does not include monitoring of the effluent at the outlet of the CASS system. The teriary filters could be included in the model. However, the verification process would not allow any reasonable assessment of what performance is provided by the secondary process and what is provided by the tertiary process. During the 26th of March 2016, one sample was taken from the Intermediate Effluent Tank at 8:00 a.m. This sample is believed to be taken after the effluent was sitting for about 2 hours in the tank. The results of the sample analysis are presented in Table 16.

Table 16: Effluent testing results taken from the Intermediate Effluent Tank during 26th of March 2016 at 8 a.m.)

Description	Units	Result
Nitrate	mg/l	7.8
Nitrite	mg/l	4.1
Ammoniacal Nitrogen	mg/l	24
COD	mg/l	110
TKN	mg/l	27
Total Suspended Solids	mg/l	46

The Ammoniacal Nitrogen result shown in Table 16 is high. This high level can be a result of incomplete nitrification within the biological system and / or as a consequence of a release of ammonia from the accumulated sludge at the bottom of the intermediate tank.

Ammonia level monitored through the SCADA system during the 26/3/16 shows an ammonia level of 7.4 mg/l. This figure supports the assumption that incomplete nitrification is occurring in the main biological reactor. Generally, we would expect close to complete nitrification to occur in the Mangawhai CASS system in the off-peak season, particularly since all the three cells within the CASS system are aerated during the entire react stage.

Another effluent parameter that was high (See Table 16), is the level of TSS. We would expect the level of TSS out of the CASS system to be in the order of 10 to 15 mg/l. The reason for this high TSS level may be due to the low stability characteristic of the mixed liquor and /or due to carry over of solids that accumulated in the intermediate tank and passed into the sample. Insufficient settling of sludge leading to sludge carry over can lead to blinding of the filters used for tertiary treatment, and possibly can impact on the efficiency of UV disinfection if the transmissivity of UV is compromised due to turbid effluent.

Based on the above and the fact that no monitoring of the CASS system effluent has taken place, the validation work could not be accomplished. As the monitored operational data has not matched our expectation, we have requested that a more detailed operational data collection programme be implemented. This would investigate the bioreactor effluent, mixed liquor and the surface layer of the main reactor to receive a better picture of the actual biological processes occurring within the CASS system.

The analysis of the biomass was completed by The Wastewater Specialists on the 23rd of November 2016. The full analysis report is attached in Appendix 1 and the summary of this biomass analysis is described below:

- The observed foaming and sludge bulking were confirmed to be a result of the formation of filamentous bacteria with M. parvicella being the most common species. This bacteria causes the flocs in the biomass to be open, creating smaller (<150μm) to moderate (150 - 500μm) flocs which inhibit settlement.

- The recommendation was that aluminum-based coagulant to improve settlement and suppress formation of the M. parvicella bacteria, be dosed directly to the main CASS reactor.

- The biomass was found to contain poor diversity and an abundance of higher life forms indicating an unhealthy process. The abundance of small flagellated protozoa and the filament Beggiatoa normally associated with low DO concentrations, high food-to-microorganism (F/M) ratio, or a process that is recovering from an upset.

- No zooglea, other bacterial monocolonies or algae were observed in the sample.

- Some organic debris and dispersed bacterial growth was observed in the biomass, but neither of these were present in excessive quantities.

- The surface layer was dominated by the filamentous bacteria M. parvicella. This type of bacteria is known to cause foaming.

The biomass analysis findings confirm our suspicions. It seems that low stability occurring due to the present of filamentous bacteria is causing carry over of solids to the intermediate tank. It was also confirmed that the oxygen supplied seems to be consumed by the higher life form in the biomass to break down BOD, leaving insufficient DO for the nitrify bacteria to achieve full nitrification.

4.4 Load Carrying Capacity Assessment

Before assessing the maximum load carrying capacity of the plant, a number of operational changes were considered to optimise the operation of the Mangawhai WWTP. Table 17 and Table 18 present the recommended changes to the CASS operational cycles and aeration.

4.4.1 Proposed Operational Cycles

The Plant is equipped with two CASS reactors which operate in parallel during the peak season from December to February. Each reactor goes through a series of steps, during which aeration, mixing, settling and decanting takes place. Table 17 presents the proposed operation cycle time of the CASS system during peak and off-peak seasons, when operating two CASS basins or one.

Table 17: Proposed CASS operation cycle time

Stage	Normal condition RECIPE (min)	High inflow RECIPE (min)	High High inflow RECIPE (min)
Fill	Continues	Continues	Continues
React	240	120	60
Settle	60	60	45
Decant	60	60	45
Total Cycle	**360**	**240**	**180**

Notes:

A during the operation of two basins, two hour lag between stages will occur.

B RAS recirculation occurs continuously during all operation stages.

C WAS removal is volumetric and occurs during the decant stage only.

For normal, dry weather operation, including over peak periods, an increase to a total cycle length to 6 hours should be considered. During high flow, such as during big rain events the total cycle time can be shortened to a 4, then, in extreme cases a 3 hour cycle to suit the hydraulic conditions. The proposed aeration presented in Table 17 is applied for both single and parallel basin operation modes. It has also been suggested that a similar increase in total daily aeration period could be achieved, retaining a 240 minute cycle, but with 160min react, 40min settle and 40min decant. Our concern with this arrangement would be that, unless sludge settling is good (low SVI), the 40 minute settle period is likely to be insufficient.

During peak season, when using a 6 hour cycle time with two reactors, a 3 hour cycle lag is required. With 6 hour cycles and two reactors, there will inevitably be an overlap when both reactors are aerating together at once, for a period of one hour and it is thus necessary to have a minimum of four blowers, with a pair dedicated to each reactor.

The benefit of this change is that the aeration capacity of the plant is increased by 30% as each basin now receives 16 hours of aeration per day rather than 12 hours under a 4 hour cycle regime.

Figure 14 illustrates the recommended cycle time operation for the Mangawhai WWTP

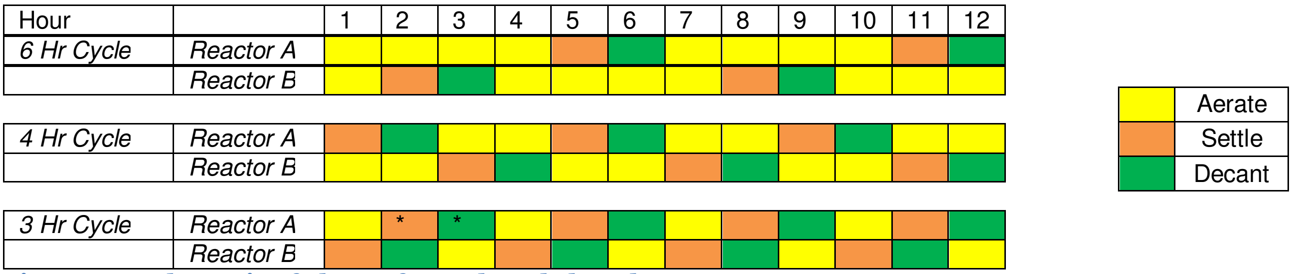

Figure 14 : Schematic of phases for each cycle length

4.4.2 Proposed "RAS" Recycle

To prevent any risk associated with short-circuiting and or increase of contaminants in the effluent caused by continuous recirculation of the RAS, it is recommended to use the RAS pumps to recycle the MLSS from the main CASS reactor into the first cell during the react stage only. This would be typical of IDEA configured plants, except that the recycle flow is normally very much higher (3-7 x Q or even more) than at Mangawhai.

4.4.3 Proposed aeration

The aeration processes taking place in the CASS system are crucial to the efficient operation of the plant. It is important that the correct balance is achieved between nitrification and de-nitrification.

Table 18 presents the proposed aeration control for each CASS system.

Table 18: Proposed aeration control

PERIOD	Normal and High Inflow Condition	
	Aeration (%) [cell1] [cell2] [cell3]	No. of "Air On" cycles (-)
React	[0] [0][100]	Alternation of aeration DO control
Settle	no aeration	
Decant	no aeration	

Notes:

A cell 1 = selector; cell 2 = secondary aeration; cell 3 = main reactor

B dissolved oxygen level set point of 2.5 mg/l use used to match current operation.

Cell 1 and cell 2 within the CASS basins were considered as designated anoxic zones as in a conventional IDEA configuration. This will create an appropriate environment for further removal of nitrogen through the denitrification process. If further removal of ammonia was found to be required, cell 2 can be used as a 'swing zone' allowing fully aerobic conditions to occur as well.

For further removal of nitrogen and phosphorus, the supply of oxygen in the main reactor of the CASS system during the react stage can be alternated. This arrangement can be valuable to enhance the removal of nitrogen in times where the loads into the plant are high, such as during Christmas / New Year – but only if there is sufficient aeration to maintain the nitrification process. To ensure appropriate contact time of the biomass and the influent and therefore efficient treatment, it is important to make sure cells 1 and 2 are mixed properly when the diffusers are turned off, as well as, when the main reactor is in react mode and the supply of oxygen is off. For this reason mechanical mixers would be required to be installed in CASS cells 1 and 2 and potentially also 3 if cyclical aeration was to be practiced there.

A further benefit of at least having Cell 1 unaerated is that it would start to function as an anoxic selector (as normally intended) and aid in the control of the filamentous organisms discussed in Section 4.3 above.

As explained in Section4.4, during the 6 hour cycle time and with two CASS basins in operation, the aeration requirement would need to be sufficient to accommodate both CASS basins working together during the overlap period occurring during the react stage for an hour. This generally means pairs of blowers dedicated to each reactor as they would both be operating at different water levels and hence blower outlet pressures.

After modification of the CASS cycle time, the RAS circulation regime and the aeration, the Biowin model was used to estimate the maximum load carrying capacity of the plant to achieve effluent levels that are still within the discharge consent limits. The following section presents the results of the load carrying capacity of the plant calculated using Christmas / New Year influent characteristics.

4.4.4 Capacity Assessment Results

As explain in Section4.4.1, to assess the load carrying capacity of the Mangawhai WWTP, Christmas / New Year influent concentrations were used as this period represent the highest loads entering the plant. To simulate the maximum capacity, the flow entering the plant was raised until the effluent requirement of the current consent limits was met. This was done while keeping in mind the expected removal capabilities of the tertiary treatment.

The maximum inflow rate to represent the maximum load carrying capacity of the Mangawhai WWTP was found to be 1400 m^3/d.

Table 19 presents the effluent results when running the model on the inflow rate of 1400 m^3/d and the comparison with the existing discharge consent limits.

Table 19: Effluent results using inflow rate of 1400 m3/d and Christmas / New Year influent concentrations

Set Up Data	Treated Effluent Quality (mg/l)		
	Parameter	Modelled Effluent at the outlet of the CASS system	Effluent Targets at the outlet of the plant
• Inflow Rate = 1400 m³/d • Wastage Rate = 22 m³/d • Recycled MLSS = 1400 m³/d • MLSS = 4500 mg/l • SRT = 20 days	TDS	-	500
	TN	28	30
	TP	8	15
	TSS	31	10
	$cBOD_5$	8	10

As shown in Table 19, the TSS value at the outlet of the CASS system is expected to be higher than the effluent TSS in the resource consent. However, this value is anticipated to be reduced to around 7 mg/l, considering the filtration system removal rate of 80%.

The following limitations and parameters used should be noted:

- The influent characteristics are based on average values collected during the Christmas / New Year period using monthly grab sample tests. This data needs to be updated by the collection of peak period, flow weighted composite samples.

- Influent Alkalinity, Calcium, Magnesium and Dissolved Oxygen were not available, therefore Biowin default values were used. It will be necessary to collect influent alkalinity data. Further, as an absolute minimum, operational dissolved oxygen levels should be monitored continuously and trended on SCADA with long term records maintained.

- No filtered COD, filtered and flocculated COD or filtered BOD_5 were available to better define the influent fractionation, therefore, assumptions were made based on recommended factors provided by the Biowin Software.

- A recycle flow from the Return Liquors Wet-Well of 140 m³ per day was used. This flow is based on 10% of the flow entering the plant as suggested by Trility to represent the filter wash water quantities. The concentrations of this flow were assumed to be similar to the final effluent as far as BOD and TP. TSS is assumed to be 80% of the TSS level out of the CASS system. Nitrogen was assumed to be similar to the final effluent plus the organic nitrogen portion captured by the filter.

- A MLSS level of approximately 4,500 mg/l was maintained by using a MLSS recycle flow of 1400 m³/d used during the react stage only and equivalent to three times the flow entering the plant and WAS flow of 22 m³/d.

- A system SRT of approximately 20 days was used to ensure the appropriate level biomass and contact time between the biomass and the influent to cope with the load increases.

- No evidence for the characteristics of flow from the dewatering system were available. Therefore the Biowin model was used to simulate the operation of the belt press and the pressate water concentration to be directed to the head of the plant.

- A DO concentration of 2.5 mg/l within the aeration cell was used to match existing operation, however this level can be run as low as 1.5 mg/l.

4.4.5 Aeration requirements

Aeration will be required to deal with the additional load entering the plant during the react stage of each of the CASS systems. During peak loading condition, where two CASS basins are in operation and 6 hour cycle time is used, the aeration requirement will need to be sufficient to accommodate both CASS basins working together during the overlap period occurring during the react stage for an hour. This is especially important if the operation of the aeration is made using one blower. However this operation is not recommended.

A preliminary sizing of the required aeration capacity has been undertaken for each of the main reactor zone and during the overlap time where both of the reactors are required to work together. The results of this sizing is presented in Table 20.

Table 20: Summary of preliminary aeration sizing and requirement for Mangawhai WWTP

Parameter	Units	Value
Maximum aeration hours per reactor per day	hr/d	16
Required air supply per reactor	m^3/hr/reactor	3,000

The number of aeration hours per a reactor shown in Table 20 represents the maximum aeration time during react stage only occurring 4 hours each cycle 4 times a day. At this study, during this react stage the aeration was turned off for 20 % of the time to ensure further removal of Nitrate.

For better efficiency and control of the required DO, it is essential that each reactor will be served by its own dedicated blower or set of blowers. Standby blowers' arrangement can be configured as per the redundancy requirement of the site.

According to the design report[18], the existing aeration system should be able to deliver maximum flow rate of 1,360 m^3/hr, however it seems that the current aeration system can provide up to 2,520 m^3/hr. This rate is calculated based on total nominal diffuser length of 252 m and diffuser rate of 10 m^3/hr/m. This rate is lower than the proposed 3,000 m^3/hr per a reactor. This difference is a result of higher loading caused by additional connected lots rate and the use of diurnal peak factor of 2.5 (refer to Table 8) to ensure that the peak hour loading is adequately accommodated.

It is unclear if the existing CASS system can accommodate the proposed air flow of 3000 m^3/hr/reactor. The As-Built diffuser set up needs to be studied to determine what air flow can realistically and safely be delivered and whether additional diffusers can be fitted in. The actual blower capacities need to be reviewed to determine what spare capacity, if any, these have. To a certain extent, these factors will dictate how much of the time that Cell 2 can be used in an anoxic mode.

[18] Mangawhai Wastewater Treatment Plant Design Report, Water Infrastructure Group, April 2009.

5 Disposal System

5.1 Background

The Lincoln Downs, Brown Road irrigation disposal site was visited by Opus Engineer Dr Marc Dresser on Monday 28th November. Marc was met on site by Robin Johnson, Technical advisor, from Trility, and then attended the Mangawhai Waste Water Community Scheme Treatment facility on Thelma South Road to meet again with Robin and also Terry Roche, Operations Manager, Trility. The aim of the site visit and subsequent meeting was to obtain an overview of the current system, the limitations and the options for increasing disposal into the future.

The information and subsequent recommendations contained in this report come from conversations and observations at these two site visits, reviewed documents and reference material, not limited to those mentioned [19] [20] [21] [22].

5.2 Lincoln Downs

5.2.1 Background

Descriptions of the collection and conveyancing system from Mangawhai to the Lincolns Downs site are contained in previous reports provided by the client.

The Lincoln Downs farm is situated on Brown Road, Mangawhai, as shown in Figure 15, and in relation to the Thelma Road Waste Water Treatment Plant, as shown in Figure 16. It was historically a fully operational dairy farm. Kaipara District Council (KDC) purchased this farm in August 2006 for the purpose of disposing of treated municipal waste water to land. The primary focus of the farm was as a productive farming enterprise. This changed in 2012 when KDC decided that waste water disposal was to be primary focus, and dry stock farming second, which allowed for a change in management priorities.

[19] Mangawhai Community Wastewater Scheme. Capacity and Upgrade Study. Final Report. 15 January 2016. Trility.
[20] Northland Regional Council Resource Consent 014969 04/05/2015
[21] URS Mangawhai EcoCare Project Assessment of Environmental Effects. Report plus appendices. August 2006
[22] Ministry for Primary Industries. Operational Code. NZCP1: Design and Operation of Farm Dairies. 1 December 2015

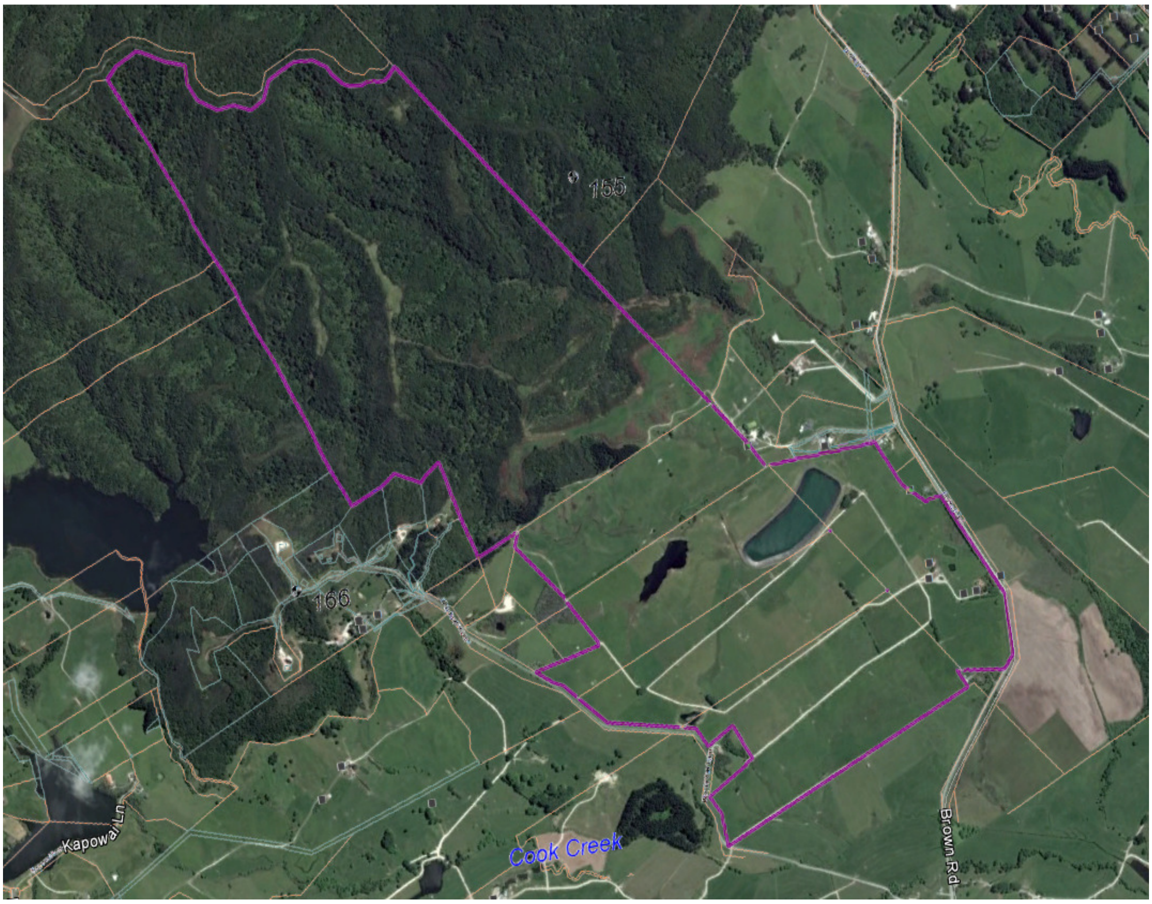

Figure 15: Map showing location of Lincoln Downs disposal site.

Figure 16: Lincoln Downs Disposal site in relation to the Thelma Road WWTP

5.2.2 Description

The Lincoln Downs farm occupies 110 ha. The wastewater holding pond, Figure 17, is 3.9 ha in surface area and holds 170,000 m³. This can be increased to 180,000 m³ if a concrete block style wave band is installed. A dedicated pumping and distribution system (Figure 18), controlled through onsite and remote PLC access (Figure 19), distributes the water to the various disposal zones.

Figure 17: Wastewater Pond

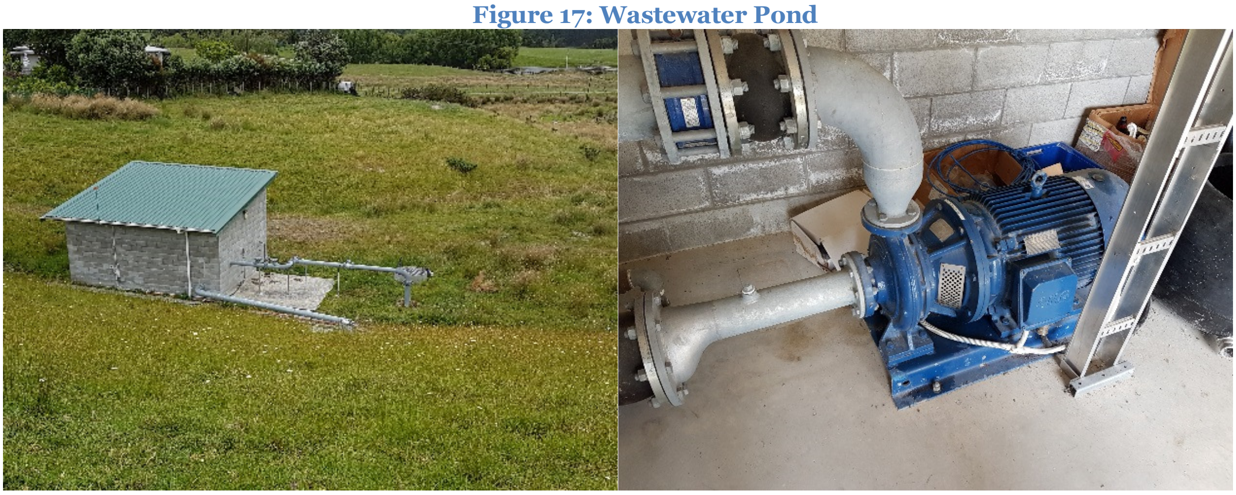

Figure 18: Dedicated pumping and distribution system

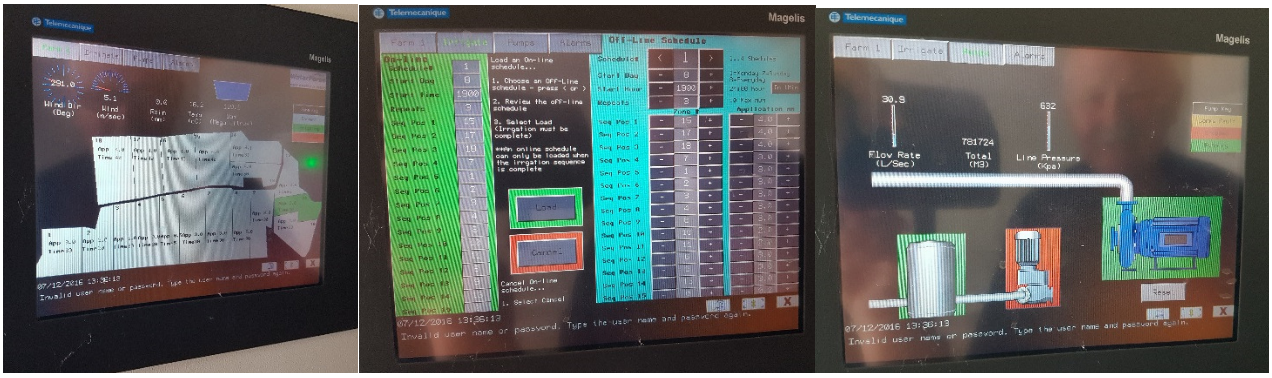

Figure 19: PLC Control

The current disposal area is shown in Figure 20 and comprises 22 zones, totalling 30 ha. The exact operation of these zones, some piggy backed and some not, is detailed in Section 2.4 of the Trility Capacity and Upgrade Study report(15 January 2016).

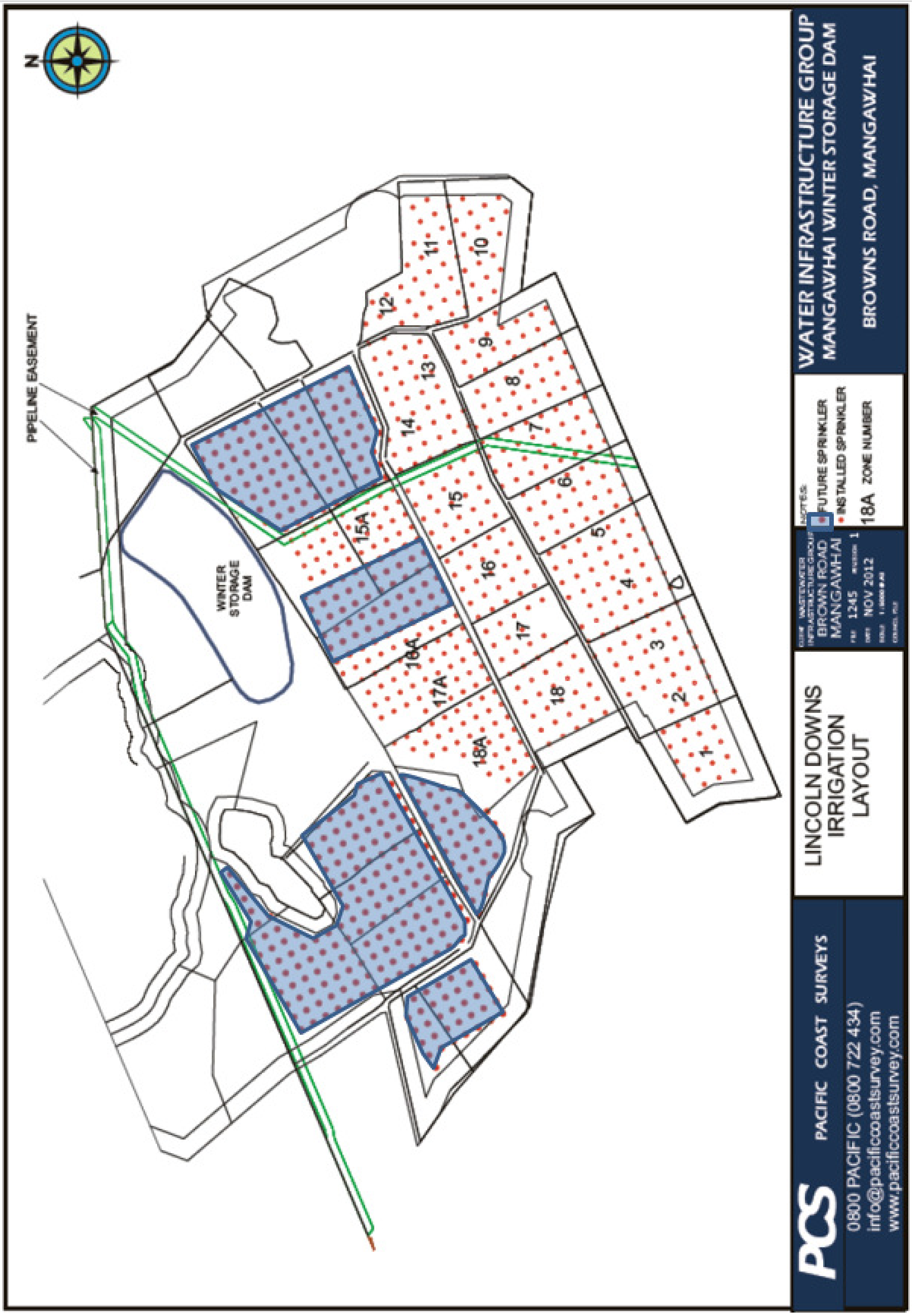

Figure 20: Current disposal area marked by red dots. Future expansion area shaded blue.

The land distribution system consists of a network of fixed sprinkler heads (Senniger) with #16 or #18 nozzles (Figure 21). These are on 20 m x 20 m grid in some paddocks and 23 m x 23 m grid in others. There are no backflow preventers or anti-siphon devices on the system. The PLC for the system is currently checked in the morning, it then cycles through its routine (irrigating a set sequence of 6 zones, 3 times in an 18 hr period). There are no alarms or alerts for operators. The system uses lockouts for climatic variables exceeding set limits, with data sourced from a nearby weather station.

Figure 21: Sprinkler Heads

The current irrigation area is approximately 30 ha. This area was inspected and the soil was found to be in good condition. There was no evidence of soil denaturing, Figure 22, pugging, or excessive waterlogging. The grass was predominantly lush and there was a good sward present.

Figure 22: Top soil sod from Zone 17A, representative of the currently irrigated area.

There is a ridge that runs along the track between the zones 15 to 18 and 15A to 18A. Zones 15A to 18A slope northward towards the pond. The slope was estimated on site to range from 5 to 15 degrees. There was evidence of surface water running over the track that separates zone 16A from the unmarked non irrigation area alongside the pond, and this is shown in Figure 23.

Figure 23: Surface water running over the track downhill of paddock 16A

This area had become waterlogged, and the animals that were in the paddock, presumed to be dairy grazing, were causing some pugging damage, Figure 24.

Figure 24: Photograph of area on the opposite side of the track from Zone 16A

If the source of the water is from the disposal area, this raise a number of concerns.

1) Irrigated waste water is finding its way onto non irrigated paddocks and causing waterlogging issues. This could be through both overland flow and underground pathways.

2) Dairy animals are grazing on this land. If the source is human waste water from the disposal area, there is an immediate food safety issue. NZCP1[23] Section 4.4, point (2) states "The application of the following wastes to land used for grazing milking animals is not permitted, a) human waste."

5.3 Expansion options

Opus were informed that there are underground pipes and riser pipework already installed on an additional 9 ha of irrigation area, as shown in Figure 20. This area was inspected and consists of land similar in slope and aspect to that already under irrigation. It was deemed that there was no immediate reason, allowing for setbacks around ponds, gully's, boundaries, tracks and water courses, that this area cannot be used in a similar manner to the existing irrigation block.

The operation of this system would rely on piggybacking these new zones due to the limitations of the existing PLC, and the current pump capacity.

When at the WWT plant, Opus were informed that there is an additional 60 ha of land that is yet to be developed for irrigation. This area has not been investigated, however a desk based study of

[23] Ministry for Primary Industries. Operational Code. NZCP1: Design and Operation of Farm Dairies. 1 December 2015

provided literature [24],[25],[26], shows this area to be suitable, and thus it is recommended that a detailed soil mapping exercise is carried out on this land to provide confidence that it is suitable to accept irrigated waste water. This detailed report should identify:

- Slope
- Aspect
- Soil type
- Limiting factors in the landscape
- Current crop and state
- Drainage characteristics, potential for runoff, and thus irrigation capacity

It is noted that the current aerial image shows the remainder of the Kaipra DC owned land to be in forestry with an undulating terrain, Figure 25. This has up to 80 m elevation change in 250 m, giving a slope of approximately 18 degrees. The suitability for irrigation will need to be checked before any decision is made on final suitability and expansion. It is also worth noting that when irrigating to forest blocks, there is generally a provision made to take any particular block out for up to 7 years for harvest prep, harvest, clearance, re-planting then establishment of the new crop to a stage where it can be safely irrigated. This needs taking into account when planning for any possible irrigation development of this area.

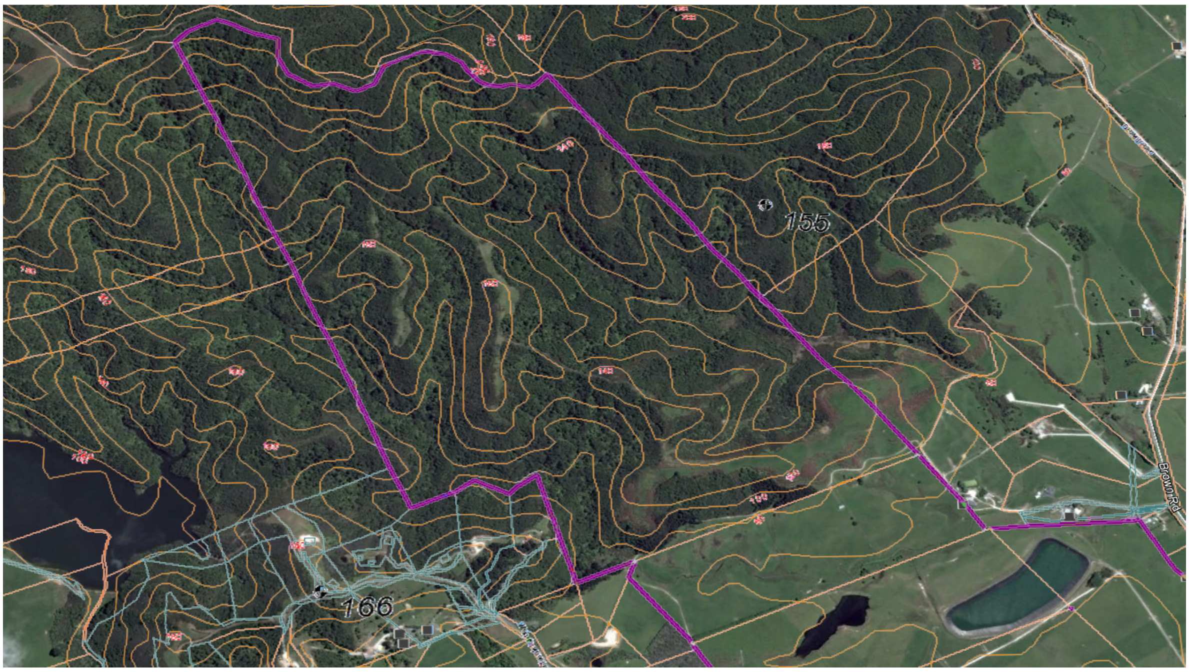

Figure 25: Contours shown in the forestry block, an area for potential expansion of disposal area.

[24] RMCG modelling report, Mangawhai, New Zealand, 18 May 2006.
[25] Tonkin & Taylor. Geotechnical investigation Report – Winter storage Dam Browns Road, Mangawhai. September 2006
[26] URS. Mangawhai EcoCare Project – Ecological Assessment of the Land Disposal Site. September 2006

5.4 Issues noted

5.4.1 Stock water

It is good practice when irrigating any effluent onto grazing land to cover the drinking troughs situated in the paddocks prior to commencing irrigation so that the effluent cannot contaminate the stock water. No covers were observed on the troughs in any of the irrigation zones.

5.4.2 Pasture condition

Some of the paddocks observed (zones 10, 11, 12) should be considered for pasture restoration. There are many non-desirable species and state of the pasture is poor. It maybe that a regime of topping paddocks to control undesirable species is a legitimate solution in the short term, however due to the paddock slopes, the soil moisture and the irrigation scheduling, this could be problematic.

5.4.3 Nutrient removal

When irrigating continuous waste water, there is a need to remove the herbage growth, in order to remove the nutrients from the system. Although grazing by beef cattle does remove some of the nutrients, it also provides for the recycling of nutrients in a semi closed system through animal excreta. Therefore it is recommended that an alternative method for removing these nutrients from the system needs to be investigated.

5.4.4 Soil moisture

Continuous irrigation often leaves the soil in an extremely moist state. When using heavy farm machinery to remove the herbage and thus nutrients from the systems, difficulty is frequently encountered when operating on slopes when the soil is moist. Curtailing irrigation to certain zones a number of days prior to using heavy machinery in that zone can improve traction and stability. As this site is in constant operation, cycling zones out of operation may result in the cutting and baling of one zone at a time, which would likely not be a cost effective operation. Other waste water disposal sites around New Zealand operate irrigation systems and still remove the herbage efficiently. It is noted that some of these, for example Taupo, Rakaia and Leeston, are all on very free draining soils, these being either pumice or alluvial gravels, and these soil characteristics will determine the level of management and operations that can occur on the blocks. It is recommended that some of these operations are approached and their experience used to improve the current system.

5.4.5 Irrigators

During the course of the site visit, some irrigators on Zone 13 were observed irrigating directly onto the track. This is not regarded as best practice and Opus therefore recommend that this is addressed.

5.4.6 Real time alarm

During the site visit, Opus were informed that the real time alarm system was not operable, and any malfunction of the system would only be picked up during routine morning remote computer checks. It is recommended that the real time alarm system is made operable.

5.4.7 Liner integrity

A number of small puncture holes were noted in the liner, it is recommended that the repair of these is actioned in the immediate future.

5.4.8 PLC clock

It was noted after studying the photographs taken during the site visit that the date and time on the irrigation PLC appears to be incorrect.

PLC display – 13:36, 07/12/2016

Actual date – 09:01, 29/11/2016

The impact of this on reporting, irrigation scheduling etc is not known.

6 Conclusion

This section presents the findings of this report to answer the scope of work to provide an evaluation of the process and hydraulic capacities of the Mangawhai WWTP while identifying whether there are any operational improvements that can be made to enlarge the capacity of the plant.

6.1 Process

Before determining the maximum carrying loading rate that the Mangawhai WWTP can treat, some modifications for the operation of the CASS system were made, these were:

- Conversion of the first and second cells within the CASS system to dedicated anoxic zones. This change was made to enhance the removal of nitrate leading to a lower total nitrogen level at the effluent.

- Increase of the cycle time from current 4 hours to 6 hours during the normal operating conditions, and shortening this time as flows increase. The 6-hour cycle will allow longer react time for further removal of contaminants.

- To improve the removal of nitrogen (nitrate), the current continuous "RAS" recycle was replaced with a 4 hour recycle time of the mixed liquor suspended solids (MLSS). This MLSS recycle will only occur during the react stage and was set to be 300% of the ADF entering the plant from the Mangawhai community. For further removal of nitrate, additional denitrification can be achieved by cycling the aeration on and off during the react stage. In this investigation, 25% of the non-aeration time during the react stage was allowed for.

Based on the evaluated maximum flow rate of 1,400 m^3/d presented in Table 19 and the influent discharge rate per connection of 650 l/connection/day (refer to Section 3.2.1), Table 21 shows the estimated number of connections that can be introduced into the Mangawhai existing WWTP, while maintaining the current effluent target specified in the plant resource consent (refer to Section 3.4) .

Table 21: Estimated number of connections that the current Mangawhai WWTP can serve

Estimated maximum flow entering the plant from the Mangawhai community (m^3/d)	Estimated Discharge Rate per a Connection (l/connection/day)	Estimated number of Connections (-)
1400	650	2,153

The number of connections of 2,153 concluded from this investigation is **162 connections** higher than the estimated current number of connections of 1,991.

Due to the increase of load entering the plant, the existing aeration system should be sized to deliver an air flow of approximately 5,000 m^3/hr per reactor. Each reactor should be fed by a dedicated blower to ensure higher DO control as well as prevent any under aeration issues occurring during the times where both reactors need to be aerated simultaneously.

6.2 Hydraulic

The hydraulic parameters used to configure the CASS operation, so it can be used to treat the Christmas / New Year concentration at a flow of 1,400 m^3/d, were compared with the findings of the hydraulic investigation presented is Table 4, to evaluate what are the process unit elements that require an upgrade. The results of this comparison are presented in Table 22.

Table 22: Process requirement and existing hydraulic capacity comparison

Description	Estimated Hydraulic Capacity (l/s)	Process Requirement (l/s)
Screen	70	74
Splitter Box	160	74
CASS Decanter System	200	171
RAS pump	8.5	24
WAS pump	14	1.5
Intermediate Transfer Pump	48	24
Filtration System	26	24
UV system	26	24
Belt Press System	4.16	0.25

The following information was used to evaluate the different process flow requirements shown in Table 22.

- To determine the flow requirement of the screen and the splitter box, the estimated inflow of 1.400 m^3/d was divided by the peak factor of 2 to reach the annual daily average (peak factor of 2 was found to be the difference between ADF to Christmas / New Year flow, refer to Section 3.2.2). Once the annual flow was determined, the instantaneous peak factor of 8.75 that was established in Section 3.2.3, was applied to determine the peak instantaneous flow. Once the instantaneous peak flow was determined, the filter wash water of 140 m^3/day, the sludge de-water wash water of 30 m^3/d (as suggested by Trility) and 80 m^3/d of reject water were added.

- The CASS decanter flow is based on only one basin decanting at a time during peak wet weather flow, while the cycle time is reduced to 3 hours.

- The RAS pump flow is based on a recycling rate of 4,200 m^3/d, which is 3 times the inflow rate to the plant, pumping during the react stage for only 4 hours, 4 times a day.

- The WAS pump flow is based on a wastage rate of 130 m^3/d, pumping during settle stage for 1 hour, 4 times a day.

- The Intermediate Transfer Pump, Filtration System, and UV system are based on peak wet weather flow of 2100 m^3/d, which is 1.5 times higher than the average flow over Christmas/ New Year (refer to Table 7).

- The wastage rate to be de-watered by the belt press system is based on 22 m^3/d. This flow will be directed to the sludge storage tank and be fed to the belt press system at an optimum dry solids rate of 60 to 80 kg/hr.

Based on the above, the screen and the RAS pumps will require an upgrade to serve the increased inflow rate of 1,400 m^3/d. The replacement of the screen could be accommodated by adding a second screen and using the two in a Duty / Assist arrangement.

6.3 Disposal System

The site visit and subsequent literature reviewed demonstrated that there is a lot of extra capacity at the farm for receiving waters in 3 areas in addition to the existing 30 ha:

1) In the 9 ha with existing infrastructure

2) In the 60 ha that has already been identified by Trility staff

3) Approximately 100 ha of forestry block.

Option 1 can be undertaken using existing PLC and pump, although this would entail piggy backing a number of zones.

Option 2 would require an additional PLC to control the new zones, and additional infrastructure and pump to deal with the increased flowrate.

Option 3 has not been investigated.

The planned additional connections to the system would mean an increased flowrate, and thus the suitability of the main line, from the WWTP to the Lincoln Downs holding pond, which can sustain 70 L/sec flow, is unknown. If dry weather peak flow is estimated at 630 L / connection / day, current connections are approximately 2000, and with growth is expected to reach 7000.

This suggests, through basic mathematics, that currently the pumping requirements are 1.26 mil L per day, which is pumping for 5 hrs per day. If growth targets are reached, pumping requirements would increase to 4.41 mil L per day and a pumping time of 17.5 hrs per day.

Thus based on this crude calculation, the pipe infrastructure is in place to cope with the growth, but the disposal system needs increasing.

7 Discussion

This section presents several points for discussion, these are:

- The number of connections the Mangawhai Wastewater Treatment Plant serves is limited by the loads during Christmas and New Year. The Christmas and New Year period only covers about 3 weeks of the year. The Mangawhai Wastewater Treatment Plant capacity can be larger if it was operating to treat the annual average loading rates. Options to mitigate the impact of the Christmas/ New Year high-loading period could be discussed so further treatment capacity can be allowed for. For example, is it feasible to sacrifice the nitrogen removal functionality during this period and still comply with annual compliance limits.

- This report is based on a monthly grab sample influent analysis which does not provide a good representative indication of what is entering the plant. Furthermore, due to no sampling, no indication of the current CASS system performance and characterisation of the return liquors from the wash water and dewater systems were established. To be able to establish a more accurate performance capacity review of the Mangawhai Wastewater Treatment Plant, an improved monitoring regime should be put in place. The results of this improved monitoring regime can also be used as a basis for any additional upgrade work if it was required.

- There may be some case to be made for splitting the 'point of compliance' between the treatment plant and the outlet from the storage lagoon. However, more operational data would be required to establish a basis for this.

- Literature review suggests that 60 ha of land is suitable for the discharge of treated effluent. A detailed soil mapping exercise could be carried out on this land to provide confidence that this land is suitable to accept irrigated wastewater as well as assist in the determination of the different feasible disposal method that can be used.

Appendices

Appendix 1: Biomass Analysis Report

The Wastewater Specialists

Optimising your assets

Client: Opus

Sample location: Mangawhai SBR Biomass & Foam

Sample number: 16/227 & 228

Process description: Continual feed SBR

Sample date: 23rd November 2016

Observation date: 25th November 2016

Disclaimer: While The Wastewater Specialists Ltd. has exercised their professional judgement in analysing the biomass sample provided by the Client, the liability of The Wastewater Specialists Ltd. to the Client shall in no circumstances exceed the contracted fee for this analysis.

Summary:

The filamentous bacteria *M. parvicella* was very common in the biomass, was impacting on floc structure, and is known to cause both foaming and sludge bulking (poor settlement). Given the abundance of this filament, the reported foam on the surface or the SBR and solids carry-over in the decant are unsurprising. While several methods of chemical treatment for *M. parvicella* bulking are available, our recommended method is the use of aluminium-based coagulants. This adds weight to the flocs to improve settlement, binds dispersed material into the flocs, and helps to suppress *M. parvicella*.

The biomass contained poor diversity and abundance of higher life forms, which is indicative of an unhealthy process. The abundance of small flagellated protozoa and the filament *Beggiatoa* suggest low DO concentrations, although the reported decant ammonia concentration (<0.5 mg/L) indicates sufficient DO is available for nitrification.

Recommendations:

Based on this biomass analysis and our minimal knowledge of the Mangawhai WwTP, we recommend consideration be given to:

- Slug dosing an aluminium-based coagulant directly into the SBR. While the actual dose rate should be confirmed by jar tests, the required dose is likely to be in the range of 0.10 – 0.15 litres of alum per cubic metre of SBR volume per day.
- If an aluminium-based coagulant is dosed, the pH of the SBR should be monitored closely. It may be necessary to also dose an alkaline substance such as soda ash, lime or sodium hydroxide to maintain an appropriate pH in the SBR of 6.5 to 7.0.
- Checking the control philosophy of the react phase of the SBR cycle to confirm whether this is fully aerated, or whether it includes some unaerated (anoxic) periods.
- Checking the DO concentration during the aeration phase of the SBR cycle.

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- Checking ammonia and nitrate concentrations in the decant to confirm the reported concentrations of <0.5 and ~3 mg/L.
- Submitting biomass samples to us for analysis approximately once per sludge age, or at least once per month. When both SBR's are on line, a sample of the biomass from each SBR should be submitted for analysis. This ongoing biomass analysis will allow the effects of process changes on the health of the biomass to be monitored, and it will also provide valuable information for avoiding or troubleshooting potential future problems.

Analysis:

The samples were analysed using brightfield and phase contrast microscopy at magnifications of x100, x400 and x1000. Gram and Neisser stains were both undertaken to assist with filamentous bacteria identification. Abundance of filamentous bacteria was determined using the subjective scoring index suggested by Jenkins *et al*. (2003).

Typical floc characteristics – Biomass

Flocs were small (<150µm) to moderate (150 - 500µm) in size, and were diffuse due to the effects of filamentous bacteria (see photo right). Filaments were causing the flocs to be open, and were also causing some inter-floc bridging.

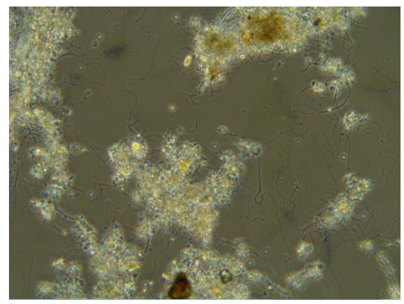


Photo: General floc characteristics, x100 magnification, phase contrast

Filamentous bacteria – Biomass

Filamentous bacteria were very common in the biomass (4/6). The dominant filament was *M. parvicella*, with Nocardioforms also being common. Other filaments observed included *Beggiatoa*, Type 0041/0675, Type 1863, and *H. hydrossis*.

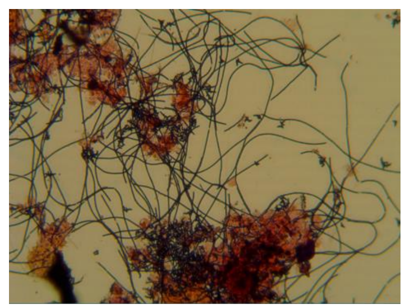


Photo: M. parvicella and Nocardioforms, Gram stain, x400, magnification, brightfield

Higher life forms – Biomass

The diversity and abundance of higher life forms was poor. Small flagellated protozoa were the most abundant group of organisms, and were very common. Otherwise, small numbers of stalked ciliates, swimming ciliates and occasional naked amoeba were all that were observed.

Photo: Stalked ciliates, enlarged from x100 magnification, phase contrast

Bacterial or algal colonies – Biomass

No zooglea, other bacterial monocolonies or algae were observed in the sample.

Other features – Biomass

Some organic debris (see photo right) and dispersed bacterial growth was observed in the biomass, but neither of these were present in excessive quantities.

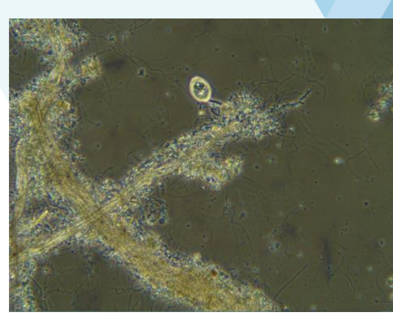


Photo: Organic debris, x100 magnification, phase contrast

Other features – Foam

The foam was dominated by the filamentous bacteria *M. parvicella* (see photo right).

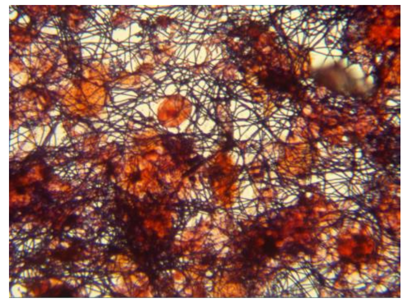


Photo: M. parvicella,
Gram stain, x400 magnification, phase contrast

Biomass Trends:

The biomass characteristics from our analysis of the Mangawhai SBR biomass are summarised in the following tables.

	23-Nov
Flagellated protozoa	2 (+++)
Swimming ciliates	1 (5)
Crawling ciliates	
Stalked ciliates	3 (+)
Naked amoeba	(1)

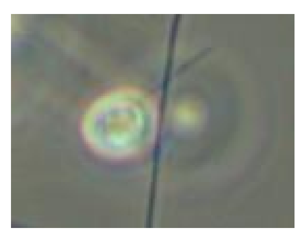
Key to previous table:
The first number denotes the number of different types observed from each group of organisms (the diversity)
The number in parenthesis indicates the total number of organisms observed in each group (the abundance)
+ =>10
++ =>25
+++ =>50

	23-Nov
Overall filaments (/6)	4/6
〉 *M. parvicella*	4
〉 Nocardioforms	3
〉 *Beggiatoa*	1.5
〉 Type 0041/0675	1
〉 Type 1863	1
〉 *H. hydrossis*	1

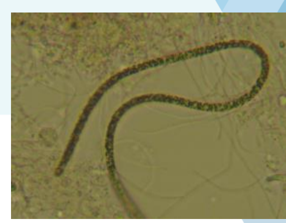
Discussion:

From our brief discussions with Robin Johnson (Trility) and Asaf Rachmani (Opus), our understanding of the Mangawhai WwTP is as follows:

- It is a continuously fed sequencing batch reactor (SBR) process, with a single SBR is use for ~11 months of the year and two SBR's operating over the peak Christmas/New Year holiday period. The plant is currently operating on a single SBR.
- The normal cycle is 6 hours in total, with 4 hours react, 1 hour settle, and 1 hour decant. During high flows, the cycle drops down to a 3 hour cycle, with each phase of the cycle reduced proportionally.
- The react phase of the cycle is fully aerated to achieve a target dissolved oxygen (DO) set-point of 2.0 – 2.5 mg/L. No anoxic (unaerated) phase is built in to the react phase.
- Current mixed liquor suspended solids (MLSS) concentration is ~1,600 mg/L, calculated from on-site total suspended solids (TSS) measurement of 0.08 g in 50 mL of sample.
- The 30-minute sludge settlement is 500/1,000 mL. This gives a sludge volume index (SVI) of ~300 mL/g, indicating poor sludge settlement.
- The ammonia and nitrate concentrations in the decant were <0.5 and ~3 mg/L at the time the samples were submitted to us for analysis.
- The surface of the SBR is covered in foam.
- Some solids carry-over in the decant may be occurring, but we do not know the extent of this solids carry-over.
- Tertiary treatment includes filtration, with the chlorinated backwash from these filters returned to the head of the plant.



It should be noted that this is the first time we have analysed the biomass from the Mangawhai WwTP so we don't know what is normal for this plant, however from this analysis the biomass appears to be unhealthy. The diversity and abundance of higher life forms was poor. Small flagellated protozoa (see photo right) were the most abundant group of organisms. Flagellates are associated with low DO concentrations, high food-to-microorganism (F/M) ratio, or a process that is recovering from an upset.



Other indicators of adverse conditions were also observed, such as the presence of the motile filament *Beggiatoa* in moderate concentrations. Due to its ability to utilise hydrogen sulphide, this filament is strongly associated with septic or low DO conditions. This septicity or low DO could be in the treatment process itself, or it could be in the reticulation. The photo right shows the distinctive Sulphur granules in a *Beggiatoa* filament which result from its utilisation of hydrogen sulphide.

Given the abundance of flagellates, the presence of *Beggiatoa*, and the general absence of other higher life forms, we are a little surprised that the plant is currently achieving full nitrification (conversion of ammonia to nitrate). We would expect a fully-nitrifying SBR process with a fully aerated react phase to contain a much greater diversity and abundance of higher life forms. If, as advised by Robin Johnson, the react phase of the SBR cycle is fully aerated, the low concentration of nitrate in the nitrified effluent is consistent with our observations in this biomass analysis, and suggests low DO concentrations are being achieved. For the decant to be low in both ammonia and nitrate when the react phase is fully aerated suggests that simultaneous nitrification and denitrification are occurring during the react phase. For simultaneous nitrification and denitrification to occur in this way, the actual DO concentration in the react phase must be significantly lower than the advised target DO of 2.0 – 2.5 mg/L. If the DO concentration was consistently 2.0 – 2.5 during the react phase, minimal denitrification (conversion of nitrate to nitrogen gas) would be expected to occur.

We must note it is possible that we misinterpreted the information given to us by Robin Johnson. If the react phase does contain both aerated and unaerated (anoxic) periods, this would be more consistent with conventional nitrification and denitrification occurring during the different parts of the react phase.

Filamentous bacteria were very common in the biomass, and were impacting on floc structure through both inter-floc bridging and by causing flocs to be open. The dominant filament was *M. parvicella*, closely followed by Nocardioforms. Both of these filaments cause foaming, therefore it is no surprise that foam is currently on the surface of the SBR. In addition to causing foaming, *M. parvicella* can also have a significant adverse effect on sludge settlement. This filament is listed in Eikelboom (2000)'s list of five filaments which can have a large impact on SVI. Therefore, it is no surprise that the SVI is currently high, with an SVI of ~300 mL/g indicating significant sludge bulking. Such bulking is likely to result in solids carry-over in the decant, particularly with a relatively short settlement phase of 1 hour.

The most common causes of growth of *M. parvicella*, as summarised by Jenkins *et al* (2003), are:

- Long sludge age, low food-to-microorganism (F/M) conditions
- Low dissolved oxygen (DO) concentrations
- Low temperature
- Presence of anoxic, anaerobic and intermittently aerated zones

In addition, *M. parvicella* is widely associated with the presence of fats, oils & grease (FOG) in raw wastewater (Richard, 2003; Gerardi, 2006; Eikelboom, 2000), and has also been associated with fatty acids (Neilson *et al*. (2002)).

Given other observations in this biomass analysis, it is likely that long sludge age and/or low DO are the primary cause(s) of the growth of this filament at Mangawhai at this time. Normally when *M. parvicella* is abundant due to long sludge age, the rate of waste activated sludge (WAS) can be increased to reduce the sludge age. However, in this case, the MLSS concentration is already relatively low, so it may not be feasible to increase sludge wasting without reducing the MLSS to dangerously low levels. The current low MLSS at a probable long sludge age is likely just a function of the low wastewater load at Mangawhai at this time of year.

Bulking due to *M. parvicella* can be treated chemically. Options include chlorination to kill filaments, the use of polymers to aid settlement, and the use of aluminium-based coagulants to enhance settlement by adding weight to the floc. Aluminium-based coagulants, such as aluminium sulphate (alum) or polyaluminium chloride (PAC) have also been found to suppress the growth of *M. parvicella* (Jenkins *et al*. (2003)). Our preference for chemical treatment of *M. parvicella* bulking is the use of aluminium-based coagulants, and we have recently had good success with such an approach at another WwTP which had significant sludge bulking due to *M. parvicella*.

According to Wanner (1994), alum dose rates of 5 to 6 mg aluminium (Al) per litre of influent flow for continuous dosing, or 15 to 20 mg Al per litre of aeration tank volume for intermittent dosing, are likely to be effective dose rates. The optimal dose rate could be determined through simple jar tests. Dosing the alum into the SBR is a simpler method for a short term fix because this can be manually dosed periodically, rather than requiring a dosing pump. We do not know the volume of the SBR, so cannot provide likely dose rates. However, based on our recent dosing experience at another WwTP, the required dose rate is likely to be in the order of 0.10 – 0.15 litres of alum per cubic metre of SBR volume per day.

It is unknown at this stage how long it will be necessary to dose alum into the SBR. Literature suggests at least 10 days of intermittent dosing would be required (Wanner, 1994), however in our recent experience dosing over a much longer period (~2 months) was required. It should be noted that dosing alum may cause the pH in AT2 to drop, so it may be necessary to also dose an alkaline substance such as lime, soda ash or sodium hydroxide to maintain an appropriate pH of ~6.5 to 7.0 in the SBR.

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