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MEASURING THE POWER OF DISINFECTION: WHAT YOU NEED TO KNOW

Paper Presented by:

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Simmonds & Bristow

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MEASURING THE POWER OF DISINFECTION: WHAT YOU NEED TO KNOW

Mario Ruckli, *Graduate Engineer,* Simmonds & Bristow

ABSTRACT

In the context of potable and pool water, effective disinfection is perhaps the most important thought, and the effects of poor disinfection can be felt by an entire community. In fact, water disinfection is a concept that dates back to 2,000 BC (granted, our technology has improved to this day). As such, the operator should have a keen understanding of how to measure the power of disinfection in water. Two options for assessing disinfection power include monitoring of free chlorine, and Oxidation Reduction Potential (ORP). The two measures are different, but how do they differ and what do the numbers actually tell us?

This paper seeks to give operators food for thought and some healthy curiosity when they next find themselves measuring free chlorine or ORP. From the basics of how the humble chlorine ion reacts in water, to the reason why a millivolt is a relevant unit of measurement for water quality, the discussion in this paper is a comparison of the two monitoring methods.

KEY WORDS: disinfection, chorine, ORP, oxidation reduction potential, potable water, pools.

1.0 INTRODUCTION

Operators bear a great deal of responsibility in ensuring that water for public use is safe. To ensure safety, a range of operational targets need to be met. Free available chlorine and ORP targets are commonly set in water treatment plants and public pools, to measure the ability of the water to be disinfected.

In this paper, we will look at what the two measurements actually mean, and what we should be considering when assessing disinfection power.

So, what is *free available chlorine*?

Free available chlorine (FAC, also known as free residual chlorine) is the portion of chlorine that remains available for disinfection, after the chlorine reacts with compounds in the water. A minimum free available chlorine concentration is required in potable and pool systems as it confirms that disinfection occurred, and there is remaining chlorine to prevent recontamination.

Chlorine can react with organic and inorganic compounds, including bacteria, metals, and ammonia. Chlorine which reacts with ammonia (to form *chloramines*) still possesses the ability to disinfect water, but to a weaker extent.

FAC is comprised of hypochlorite ions (OCl^-) and hypochlorous acid (HOCl), which are formed when chlorine is added to water. The mechanism of chlorine disinfection is *oxidation*, a process of electron transfer that can destroy biological organisms. The cell walls of bacteria hold a negative charge, which causes hypochlorite ions to be repelled from the organisms, so disinfection occurs best with hypochlorous acid, since it does not have the negative charge that hypochlorite ions have.

The balance of hypochlorite ions and hypochlorous acid in chlorinated water depends on the pH of the water, with hypochlorous acid favouring lower pH (6.0 – 7.6), and hypochlorite ions favouring higher pH (7.6 – 10). A pH below 7.6 is ideal for disinfection, with pH in the low-7 range typically seen at treatment plants.

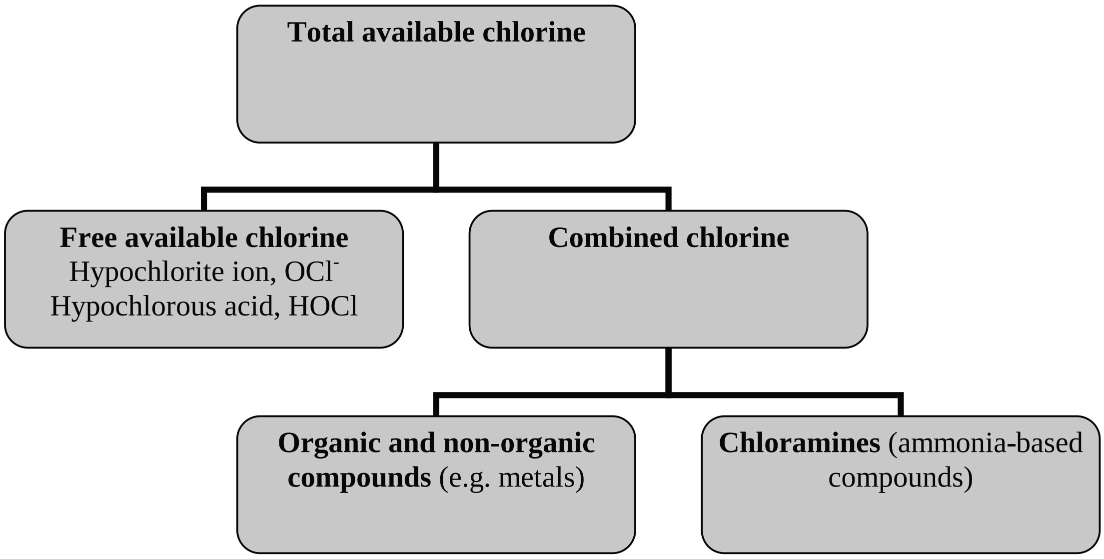


Figure 1: ***Components of chlorine in water***

What is *ORP*?

ORP stands for Oxidation-Reduction Potential, and is also known as redox potential. ORP is commonly monitored for swimming pools and potable water supplies. Oxidation was originally defined as the gain of oxygen (hence the term), however a more general definition has been produced based on electron loss and gain, with oxidation being a loss of electrons and reduction a gain in electrons. ORP is measured in millivolts and represents the tendency of a system to either donate or accept an electron.

Since chlorine disinfection is an oxidation process, ORP can inform us as to whether the conditions within the water are favourable to disinfection. A positive ORP value indicates an oxidative environment, which is desired for water treatment, whereas a negative ORP value suggests that there is a greater potential for reduction, which is less desirable.

For chlorine disinfection, hypochlorous acid is an oxidising agent. Hypochlorous acid can react with reducing agents or other less powerful oxidants, which can include hypochlorite ions, chloramines, cyanuric acid, organic matter, and micro-organisms.

2.0 DISCUSSION

2.1 Measuring Free Available Chlorine

Free available chlorine is typically measured using the DPD method. The DPD method is quick and simple, making use of DPD (N,N Diethyl-1,4 Phenylenediamine sulphate) reagents that react with chlorine to produce a magenta dye that is measured with a colorimeter.

What *does* a FAC measurement tell us?

Most importantly, a presence of FAC tells us that breakpoint chlorination has been achieved. Breakpoint chlorination is a very important idea for water treatment, and achieving breakpoint means that the chlorine demand has been fully satisfied. Any additional chlorine added to the water remains as FAC. A reasonable FAC concentration (typically 0.2 to 0.5 mg/L for potable water) also tells us that the water will remain disinfected in reticulation.

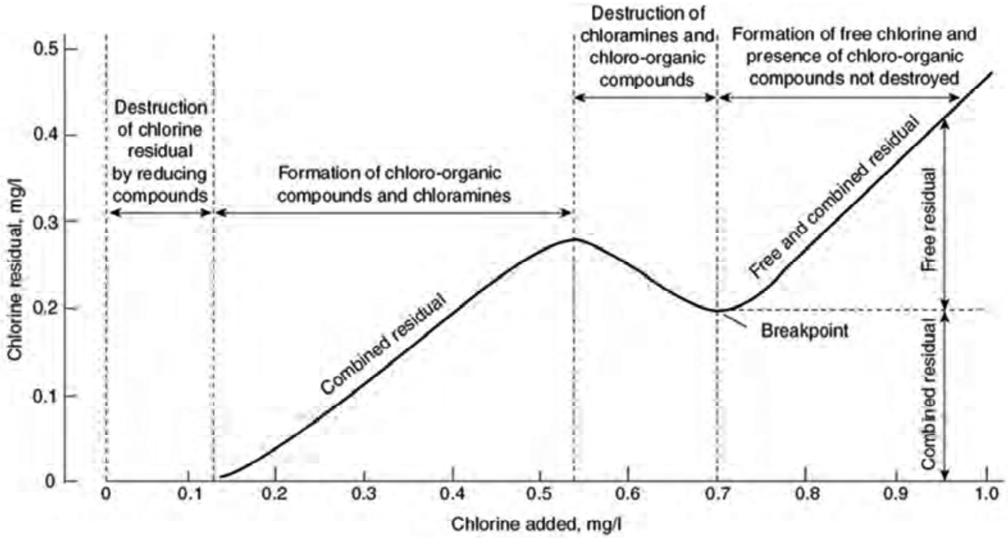


Figure 2: ***Breakpoint chlorination curve (Samer, 2015)***

What can a FAC measurement *not* tell us?

If a free chlorine residual test returns a result of zero, it is difficult to determine how much additional chlorine must be added to achieve breakpoint chlorination. Chlorine can react with a variety of materials, including organic compounds and ammonia (which forms chloramines). If ammonia is present, high chlorine to ammonia ratios are necessary for destroying chloramines to produce free available chlorine (roughly 7:1 Cl:N; chloramines will fully oxidise to chloride and nitrogen). “Burning out” the ammonia present can be a reason to carry out super chlorination in swimming pools, and can drive very high chlorine dose rates in water with naturally high ammonia concentrations.

Measuring FAC does not tell us how much hypochlorous acid we have, although we can estimate that if we also know the pH of the sample, which allows us to judge the disinfection power of the chlorine.

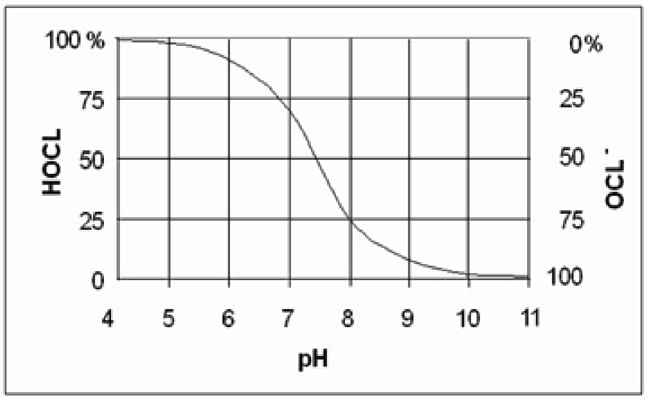


Figure 3: ***HOCl and OCl⁻ speciation for a typical pH range (Hach, 2017)***

Ideally, FAC measurement is followed up with total available chlorine (TAC) measurement (by the simple addition of DPD-3 to the sample), in order to determine how much combined chlorine is present in the sample. In fact, the Queensland Health *Swimming and Spa Pool Water Quality and Operational Guidelines 2004* suggest that combined chlorine should only form a minor part of the total chlorine – not only are chloramines far weaker disinfectants than free chlorine compounds, they are also the primary cause of taste and odour problems. Some treatment facilities do successfully apply chloramination as a disinfection strategy; however this is generally a special case as the system needs to be tightly controlled.

The presence of substantial amounts of both chloramines and free chlorine can suggest that the system is just at breakpoint chlorination.

Chemical Parameters Table

	Indoor Pool	Heated Indoor Pool	Outdoor Pool	Outdoor Pool	Spa
Water temperature		> 26^0C		> 26^0C	35^0 - 37^0 C ideal 40^0C max
Free chlorine (mg/l, ppm) minimum	1.5	2	1.5	3	3
Free chlorine (mg\l ppm) with cyanuric acid	N/A	N/A	3	4	N/A
Total chlorine (mg/l, ppm)	free chlorine level + 1 (10 max)	free chlorine level + 1 (10 max)	free chlorine level + 1 (10 max)	free chlorine level + 1 (10 max)	10.0
Bromine (mg/l, ppm) minimum	3.0	4.0	3.0	4.0	4 - 6
Ozone (for chlorine level see above)	0 residual	0 residual	0 residual	0 residual	0 residual
pH	7.2 -7.8	7.2 -7.8	7.2 -7.8	7.2 -7.8	7.2 -7.8
Total alkalinity mg/l, ppm	80 - 200	80 - 200	80 - 200	80 - 200	80 - 200
Cyanuric Acid	0*	0*	30 - 50	30 - 50	0*

***As indoor pools are protected from direct sunlight, cyanuric acid must not be used as the effectiveness of chlorine is reduced.**

NOTE: Combined chlorine shall not exceed half the total chlorine concentration with a maximum of 1.0 ppm.

Figure 4: ***Recommended chemical parameters table for swimming pools. Note the requirement for combined chlorine to be low, even though it still has disinfection power (Qld Health, 2004)***

2.2 Measuring ORP

ORP is typically measured with an ORP probe that contains an ORP electrode and a reference electrode, similar to how pH is measured.

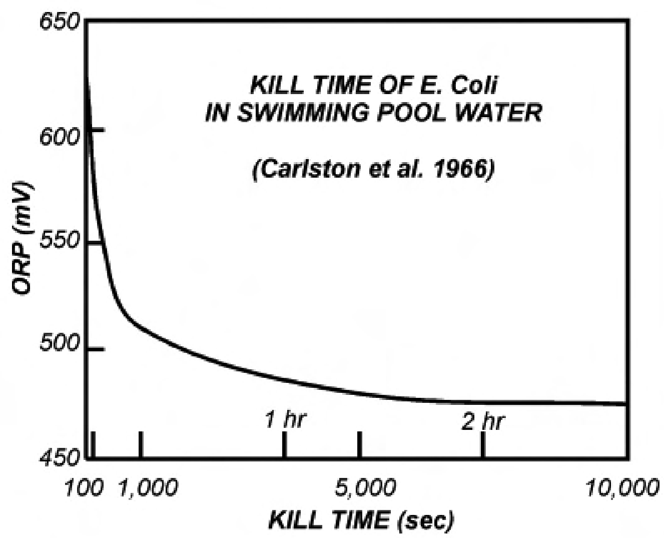
Physically, ORP is measured by comparing the output of an ORP electrode (which can either gain or accept electrons, altering its output), and a neutral "reference" electrode (which creates a constant output for comparison). The ORP measurement can be dependent upon the pH and temperature of the sample, so care should be taken to check whether your meter will automatically account for these or not.

What *can* an ORP measurement tell us?

The primary benefit of ORP measurement is that it can be an indicator of disinfection rates, with higher ORP values correlating with faster kill times of micro-organisms.

High ORP values suggest that sufficient hypochlorous acid is available in the water.

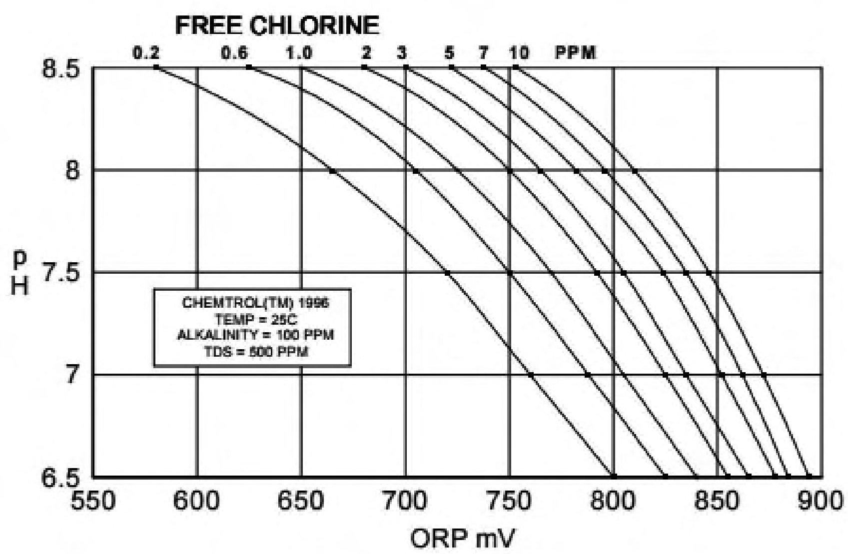
ORP monitoring is more common overseas, and suggested redox values for disinfection are available in literature. The World Health Organisation (1971) suggests a redox potential of 650 mV for drinking water. Queensland Health *Swimming and Spa Pool Guidelines* suggest redox potentials of 700 – 750 mV for swimming pools. These values do not suggest that swimming pools should be "cleaner" than drinking water – instead, higher redox potentials for pools are recommended due to the degree of contamination that can occur in a pool, requiring higher concentrations of disinfectants.



<u>Figure 5:</u> ***Kill time of E. coli for a typical ORP range (Steininger, 1985)***

What can ORP measurement *not* tell us?

ORP is not a direct substitute for free available chlorine measurement. Free available chlorine can consist of hypochlorite ions, which hold a negative charge (like many micro-organisms but unlike hypochlorous acid), and can actually decrease ORP readings – despite still having a degree of disinfection power. This issue will occur at higher pH values, and can be seen in Figure 6.



<u>Figure 6:</u> ***Free chlorine vs ORP, for a range of pH values (Steininger, 1998)***

Reiterating that ORP is the measurement the tendency of a sample to either donate or accept electrons, ORP could be high despite low concentrations of hypochlorous acid, if the balance between the oxidising and reducing agents in the water is off or other oxidants are present. Therefore, a high ORP reading cannot guarantee that a disinfection buffer will be available for water flowing into reticulation.

Similarly, ORP could be low at high concentrations of hypochlorous acid, if the water is highly contaminated. This is more of an issue for sewage treatment plants.

3.0 CONCLUSION

FAC and ORP are commonly measured in the water industry, and the results of these measurements can reveal useful information about the disinfection power of disinfectants in a sample of water. There are limitations to relying on one measurement over the other, and ideally both should be used to gain a full understanding of the disinfection power in water.

Sometimes FAC and ORP measurements require additional information to qualify the disinfection power in water, such as the pH of the water, total available chlorine concentration, or ammonia concentration.

4.0 REFERENCES

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AN OPERATOR'S PERSPECTIVE; THE FIRST NEREDA® WWTP IN KINGAROY, AUSTRALIA

Paper Presented by:

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AN OPERATOR'S PERSPECTIVE; THE FIRST NEREDA® WWTP IN KINGAROY, AUSTRALIA

Andrew Watson, *Senior Plant Operator,* South Burnett Regional Council

ABSTRACT

In mid-2016, Aquatec Maxcon along with Royal HaskoningDHV commissioned Australia's first full-scale aerobic granular sludge (Nereda®) WWTP. The plant has met all its effluent and recycled water licence requirements, but has been a significant learning curve for the operators with its 21st century technology and new instrumentation, maintenance, and testing requirements. This is an operator's perspective of the Nereda® technology and everything else that enables the plant to run.

Figure 1: ***Kingaroy WWTP 2017***

Figure 2: ***The old Kingaroy WWTP***

1.0 INTRODUCTION

The township of Kingaroy is located approximately 200km north west from Brisbane within the South Burnett region of Queensland and is administered by the South Burnett Regional Council. It is known as the peanut capital of Australia. The current population is approximately 12,500 people with 9,000 EP connected to the sewer system. The original wastewater treatment plant was constructed around 1940 with many additions in later years to support the population increase over the next 70 years. The old plant consisted of 2 primary settling tanks, 2 trickling bio filters, 2 dortman/humus tanks, a secondary clarifier, chlorine contact tank, 3 lagoons and final pond with flume to the Stuart River. The solids stream consisted of 2 digesters with mixers for the addition of lime, 1 thickener with picket fence mechanism, belt press and 7 drying beds. Time and age took its toll and the town grew. Poor effluent quality results and significant deterioration of concrete, steel work and mounting costs on mechanical infrastructure led to the need for a 21st century plant.

2.0 THE BIRTH OF A WASTEWATER TREATMENT PLANT

In mid-2014, a submission was put forward to construct a new WWTP and remove the existing plant. *Aquatec Maxcon* was awarded the tender to design and construct the new plant and reuse the settling tanks from the existing plant, commission the new plant and train operators with the new technology. The new Nereda® plant was to be the first of its kind in Australia with Dutch granular activated sludge technology.

In late 2014, works began on Australia's first Nereda® WWTP by Aquatec Maxcon with MPA Electrical and QCGC.

The new plant was designed to remove nutrient loads, bacteria, and pollutants. It would also reduce the amount of effluent that was discharged into the Stuart River and send that treated effluent back to the community as Class A recycled water, fit for irrigation of sporting, recreational and school ovals. The effluent was to be of the standard that is required for filtration and dual disinfection for reclamation.

May 2016 saw the cut over of the raw sewer main and the first influent pumped into the bio-reactors, and the plant was ready for commissioning. Eager operators took advantage to observe the commissioning of the new plant. Within days of operating, the plant met all its final effluent DEHP Licence conditions; with low ammonia, total nitrogen, total phosphorous and faecal coliforms. Except for the granulation, commissioning of the plant was well underway. The Aquatec Maxcon and Dutch commissioning engineers came to oversee the nearly completed project and were very pleased. The training covered the Nereda® system operating principles, sample collection and analysis and troubleshooting of the system should the need arise. The sample analysis focused on how to measure MLSS, SVI_5 and SVI_{30} and its comparison, granulation percentages/formulation and WAS monitoring program.

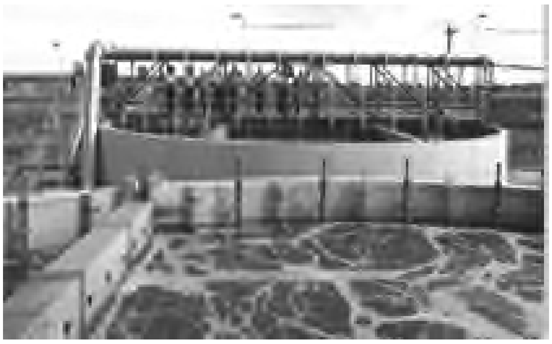


Figure 3: ***Commissioning team*** **Figure 4:** ***Aerial Photograph 2016***

3.0 NEREDA® TECHNOLOGY

Nereda® technology, or Aerobic Granular Sludge technology, has been developed, refined, and mastered by Royal HaskoningDHV with years of trials. I believe this will be the new standard in biological treatment of municipal wastewater. The process within the bioreactor manipulates and encourages the formation of activated sludge or biomass to encourage granulation and grow granules. These granules are responsible for the removal and uptake of phosphate and achieve excellent settling results. With faster settling, the granules within the granulated bio-reactors can process more influent. Another major benefit is plant footprint is at most, half the size of a conventional oxidation ditch treatment plant, resulting in significantly reduced capital expenditure.

Figure 5: ***Faster granule settling achieved with new Nereda® technology***

The liquid process flow of our Nereda® plant is as follows: we receive approximately 1.8ML of influent daily; it enters via 450mm pipe into the screening and grit removal. The influent enters the balance tank which holds the screened influent until the bioreactors call for feeding. The plant has a design ADWF of 2.62 ML/d, but is currently operating with approximately 1.8 ML/d.

There are three (3) main phases of the Nereda® process:

- **Feeding** to feed and displace the effluent at the top of the bio-reactors,
- **Aeration** to aerate/mix the biomass to nitrify/denitrify.
- **Settling** before displacement of final effluent with the start of the next feed phase.

Depending on batch size, it can be completed within three - four hours. The granules within the bioreactor are the workers. They form fast settling biomass including Phosphate Accumulating Organisms (PAO). Each granule can work or operate in anaerobic, anoxic, and aerobic conditions. These granules perform differently to any other bacteria or biomass occurring within traditional anaerobic or aerobic WWTP plants. These robust granules are right at home here in the Kingaroy WWTP reactor or any other Nereda® reactor. Each reactor is capable of MLSS of around 6000-8000mg/L and will run at peak performance. At present, the Kingaroy bioreactors are operating at 3500-6000 mg/L and increasing steadily.

When Nereda® is in 'feeding' phase the raw influent is pumped into the bioreactor displacing the final effluent from the previous batch which has a turbidity of around 1-3 NTU and is pumped to the clarifier to be stored and further polished. The Nereda® process does not require further clarification, but the clarifier was left in place to be used as storage and feed the filter feed tank. The filter feed tank supplies effluent to the filter feed pumps for the recycled water facility. When the filter feed tank is full, the treated effluent is diverted toward lagoons, disinfected with hypo with detention time around 30min, and then discharged into the lagoon and finally the flume into the Stuart River.

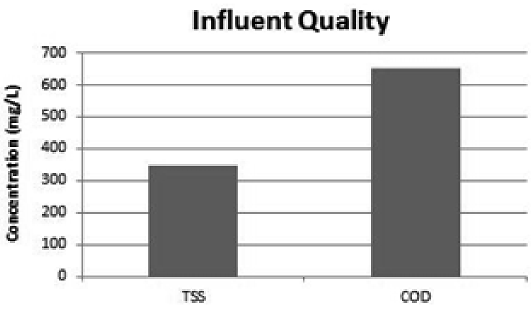
There are multiple barriers to meet Class A recycle water requirements including effluent dosed with alum and fed into pressure filters at 20 L/s, and being double disinfected by UV and sodium hypochlorite and fed into the contact tank before being stored. There are standalone pumps each supplying customers via telemetry that have a requirement for the Class A recycled water. Each pressure filter is serviced by air scour and backwash pumps and wet rack to ensure turbidity, transmissivity, and chlorination monitoring for SCADA interface.

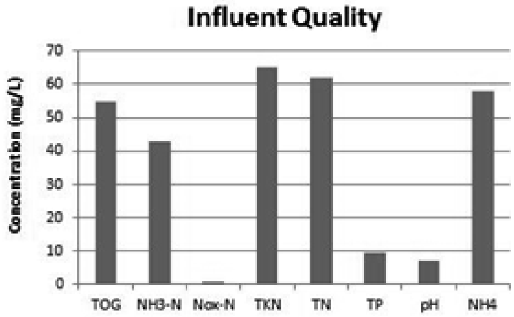
The solid streams are transferred from the bio-reactor to the WAS buffer tank. This wasted sludge is pumped to the thickener. The thickener is kept at 30,000 mg/L with the assistance of poly added at the time of wasting. The solids are pushed through the bottom via hydraulic loading of the thickener, enters the aerobic digester for a three-stage digestion and maturation prior to being pumped to the gravity drainage deck and belt filter press for disposal. The final digested sludge is around 13000 mg/L. Sludge dryness is approximately 15% with a capture of at least 91-95%.

4.0 EFFLUENT AND INFLUENT QUALITY

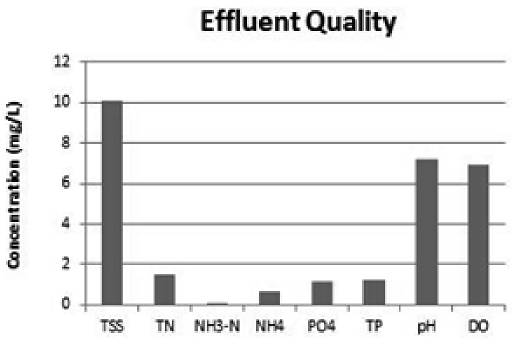
Kingaroy influent is typical domestic sewage and there are no major industries that rely on us to process their waste. The online analysers that capture data at the balance tank feeds this information to the Nereda® Aquasuite© Controller.

This allows the controller to adjust and operate to each batch that is fed into the reactors. Below are the median influent and effluent characteristics we receive at Kingaroy WWTP.





<u>Figures 6 & 7:</u> ***The median influent characteristics to Kingaroy WWTP***

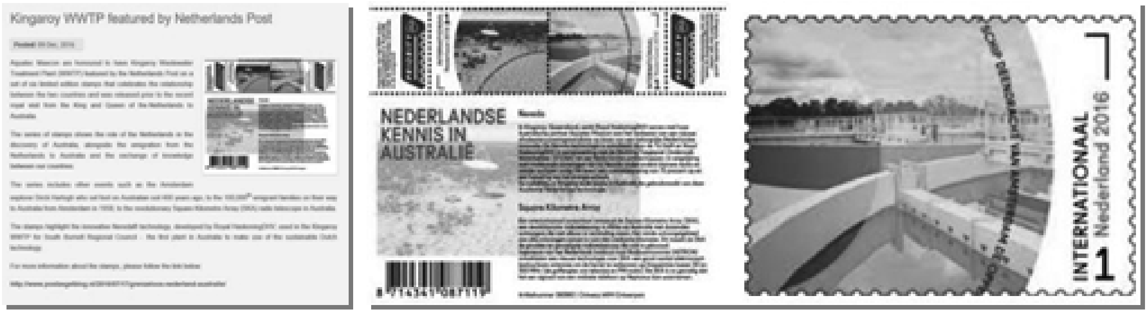


<u>Figures 8 & 9:</u> ***The median effluent characteristics and final lagoon outfall***

5.0 THE BENEFITS OF THE KINGAROY NEREDA® WWTP

- The new Nereda® Process plant footprint is at most, half the size of a conventional treatment plant, so less walking for me! It is more energy efficient compared to most aerobic treatment plants, is chemical efficient (almost no addition once full granulation occurs), is compact with no moving parts, and has a reduced amount of mechanical equipment as everything happens within one tank.
- Remote monitoring - ability to dial into the network and start/stop/operate the plant from anywhere in the world.
- Principal Contractor was actively involved in operation/commissioning/training and available to assist after hours. Royal HaskoningDHV and Aquatec Maxcon are always keen to talk to new rookies and assist with any issues.
- Operatable flexibility – ability to adjust any operable parameters.
- Availability of redundancy - with duty/standby and duty/standby/assist we now have greater flexibility when performing maintenance on equipment without having critical equipment offline.
- On-line instruments and SCADA interface to allow all instruments to be interactive with the operator and programing logic to integrate with characteristics to operate with the Nereda Aquasuite© controller to perform all controller functions including batch sizes.
- Effluent discharged from the reactor is suitable for recycled water reclamation. During summer, up to half of the influent is reused for sporting fields and a golf course. This recycled water has a turbidity of <1NTU and faecal coliforms of <1cfu/100mL.

- If power fails or there is disturbance in the power generation grid, the on-site generator will assist and continue until a reliable continual power source is returned.
- All processes for the liquid stream are started and completed within 1 bioreactor. There is lower investment and no moving parts within the bioreactor.
- Kingaroy at present has a maximum of 2.6ML a day, but should the town grow, we only require another reactor. Most of the infrastructure is in place.
- Excellent treatment with better nutrient, bacteria, and pollutant removal. Low total nitrogen (TN), total phosphorus (TP) and ammonia (NH3-N).
- Royal HaskoningDHV are monitoring the process all the time and tweaking the recipe, to achieve a more efficient plant.
- *And finally, the best thing;* ***we made it onto a stamp****!*



<u>Figure 10:</u> ***The Kingaroy WWTP made it onto a stamp by Netherlands Post – We are recognised up there with Dirk Hartog! He is on the same stamp series***

The stamps featured the relationship between two countries highlighting the exchange of knowledge and the innovative technology of Royal HaskoningDHV, to make use of sustainable Dutch technology, used in the Kingaroy WWTP for South Burnett Regional Council.

6.0 OUT WITH THE OLD AND IN WITH THE NEW

The new system requires more maintenance than previously to keep the plant running at its optimum. The following differences to the old plant are:

- There are online analysers that are required to be kept in peak condition including replacing reagents. All instruments are Hach supplied instruments – Amtax, Phosphax, Filtrax, D.O. probes, Nitrate Probes, Suspended Solids, pH probes, Ultraturb, UVT and Chlorine analyser. This equipment requires regular maintenance and calibration. The equipment is very expensive and we have arranged a service agreement with Hach. In saying that, the effluent licence is much stricter and we are passing on all criteria by a significant margin.
- Power usage has increased due to the use of blowers and pumps compared to the old trickling filter plant that was mainly gravity driven.
- Meeting the licence criteria and producing Class A recycled water for use within the community.
- If the Nereda Aquasuite© controller crashes, the system will revert to the back-up or emergency recipe. There is a high-end server with standby hard drive and CPU. As long as this is maintained there will not be issues.

7.0 OUR NEW COUSINS IN RIO DE JANEIRO (NEREDA® BROTHERHOOD)

I visited the Rio de Janeiro Nereda® plant in Deodora, Brazil, where they received very similar results. Release characteristics vary due to the environmental licence and local authorities around the world, and how regulated the country is. The Deodora plant was built in two stages - 64ML/d and secondary stage 84ML/d. This treats wastewater for 480,000 EP. Should you wish to know more about Nereda® and other treatment plants, visit me at Kingaroy or see the following websites:

www.royalhaskoningdhv.com/en-gb/nereda/plants/australia-kingaroy/4001 and

www.aquatecmaxcon.com.au/technologies/sewage-treatment/nereda-aerobic-granular-biomass

<u>Figure 11:</u> ***Google Earth Image of Deodora WWTP***

<u>Figure 12:</u> ***Foz Aguas 5 staff (Manager, Andy Watson and Senior Operator)***

<u>Figure 13:</u> ***Nereda® Reactor Bridge Deodora WWTP January 2017***

THE ROLE OF THE OPERATOR IN CHALLENGING SEWAGE TREATMENT PLANT ANALYSERS AND INSTRUMENTATION

Paper Presented by:

Mark Tosh

Author:

Mark Tosh, *Plant Operator,*

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Mark Tosh, *Plant Operator,* Queensland Urban Utilities

ABSTRACT

At the Sandgate Sewage Treatment Plant (STP) a regular testing regime was established by operators for nutrient levels in the 3 Final Settling Tanks (FSTs) as well as Mixed Liquor Suspended Solids (MLSS), settleability and Sludge Volume Index (SVI) in the bioreactors.

Regular operator sampling and analysis indicated rising ammonia (NH-3) and phosphorous (P) concentrations in the FST. This was an indication of under-aeration in the bioreactor. To help with early detection of similar events a Dissolved Oxygen (DO) check list was developed to compare results of the hand held DO probe and the infield DO probe. It was established that Bioreactor 2 had been under-aerating due to a false high infield DO reading.

This paper will demonstrate the importance of regular testing for plant performance and checking of the reliability and accuracy of field instrumentation. By the establishment of critical set points/process parameters or the establishment of quality assurance system, issues can be brought to an operator's attention before an STP's performance is adversely affected.

1.0 INTRODUCTION

The Sandgate STP is managed by Queensland Urban Utilities (QUU) and is situated on Paperbark Drive at Sandgate. It has the Boondall Environmental Wetlands in its surrounds as well as Cabbage Tree Creek into which the final effluent is released. It services the areas of the north Brisbane suburbs of Boondall, Shorncliffe, Deagon, Bracken Ridge and Sandgate. Due to the Boondall Environmental Wetlands, Cabbage Tree Creek and the close proximity to residential areas, it is considered environmentally sensitive.

Sandgate STP has design capacity to treat 25 ML/d dry weather flow, and an equivalent person of 110,000. It is a Biological Nutrient Removal (BNR) plant with two oxidation ditches/Bioreactors (BR) utilising fine bubble diffused aeration, three final settling tanks for clarification. Waste Activated Sludge (WAS) is removed directly from the BR to two Belt Filter Presses; Aluminium sulphate dosing is available to aid in the removal of P and is used on an as needed basis. Ultra violet (UV) disinfection is used on the final effluent.

<u>Figure 1:</u> ***Aerial view of Sandgate STP***

<u>Figure 2:</u> ***Fine bubble diffusers***

<u>Figure 3:</u> ***Oxidation Ditch & Final Settling Tank***

Sandgate STP comes under the Department of Environment Heritage Protection Moreton Bay "Bubble licence". The plant is subject to an ammonia release limit of 4 mg/L (as a median) and a yearly phosphorous release limit of 18,250 kg/year (equivalent to a median P release of 2.7 mg/L).

Phosphorus removal from the effluent can be achieved chemically by using metal salts for coagulation of soluble P to bind in the biomass. This is then removed through wasting, or biologically by creating an environment that supports healthy Phosphorus Accumulating Organisms (PAOs). PAOs release P in an anaerobic environment, and then remove P in aerobic conditions in greater amounts than what was released – resulting in a net P reduction.

Return Activated Sludge (RAS) and influent mix together in the anaerobic zone. Here P is released into the mixed liquor and then aerobic conditions are introduced for the removal of P. NH-3 is converted to nitrates in the aerobic environment through the nitrification process. Very low to no DO conditions are introduced (Anoxic zones) to allow the conversion of nitrates to nitrogen gas for effective de-nitrification before the clarification process.

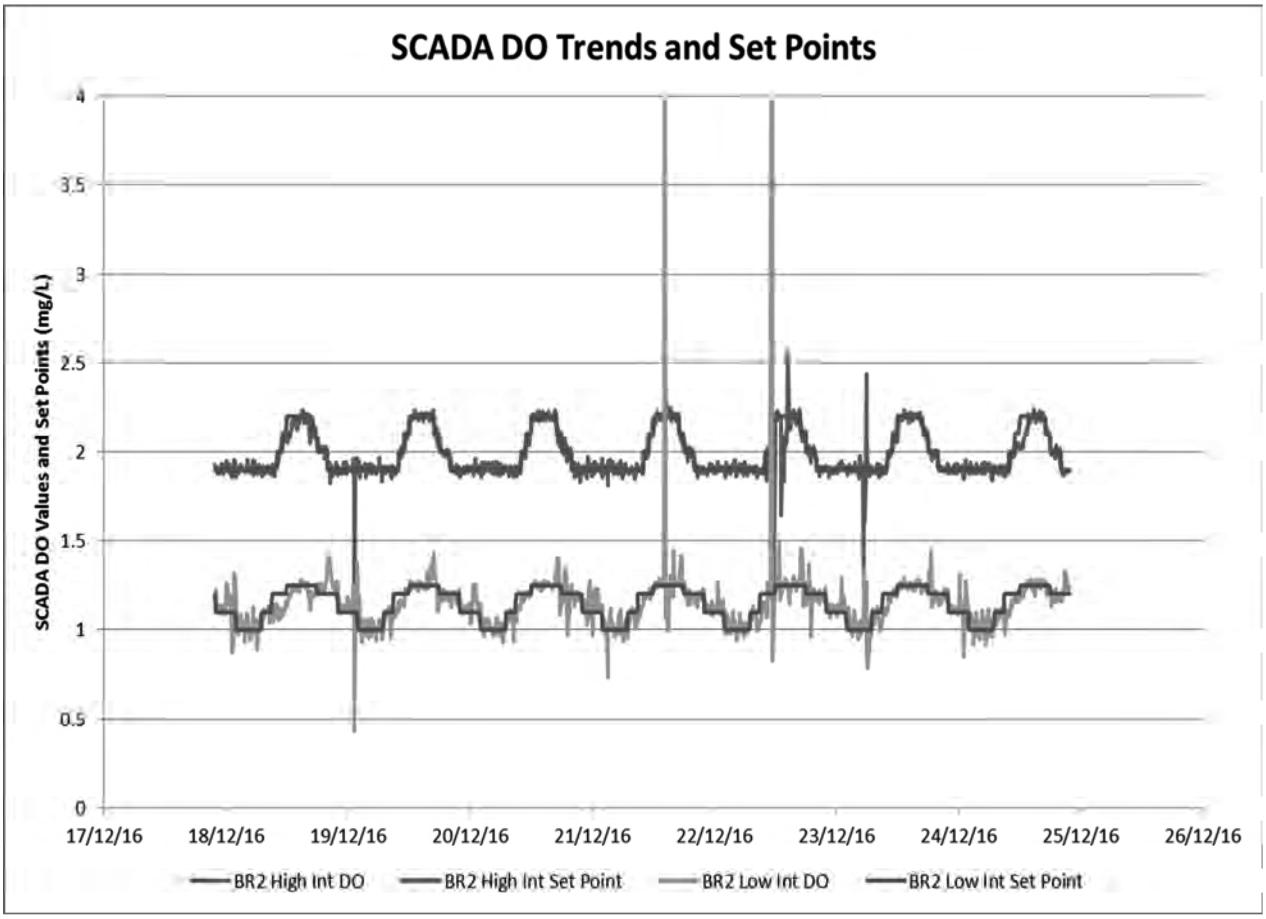
2.0 DISCUSSION

To maintain the conditions set out under the Bubble licence critical set points where established to bring attention to operational staff the need for further action if test results fell outside of the desire range, so as to bring plant performance parameters back towards a desired level of conformance.

In the past, monitoring was relied upon by in field instrumentation and scrutiny from the trends. A testing regime was introduced by operators with daily analysis of NH-3, P, N, settleability, MLSS and SVI. This information was then collated into a spreadsheet where if any of the results exceeded the desired operating envelope, further action would be required.

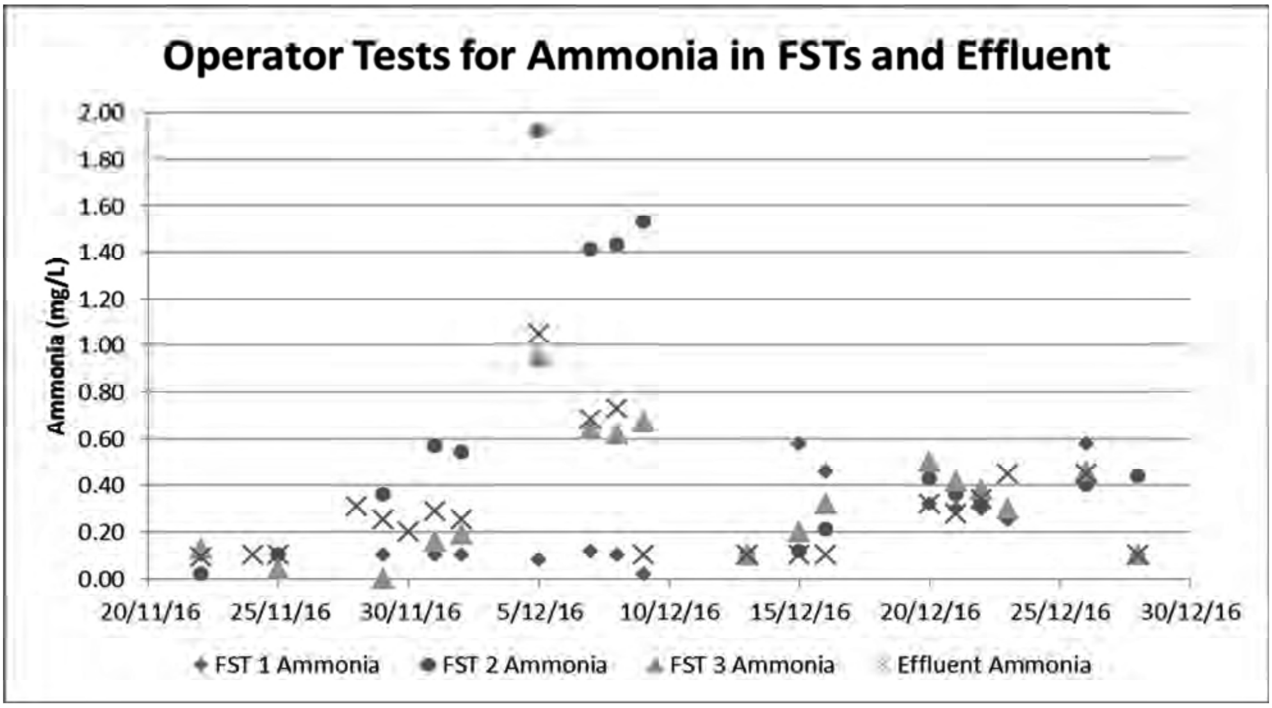
It was from this approach that it was recognised that an increase in NH-3 and P was taking place in FST 2. Although the plant had been trending correctly and meeting the desired set points, more investigation was required. Appropriate mixing in the BR's was taking place so attention was placed on the DO control.

Figure 4 shows how the DO trends were meeting the desired set points (the spikes are calibration spikes). All appears as if the plant is performing as it should.



<u>Figure 4:</u> ***DO Trends***

Ammonia spiking in FST 2 on 5/12/16 through to the 9/12/16 is shown in Figure 5. After investigation of the DO control on 10/12/16 and recalibration of the online DO instrument NH-3 dropped back down.



<u>Figure 5:</u> ***Ammonia in FSTs and Effluent***

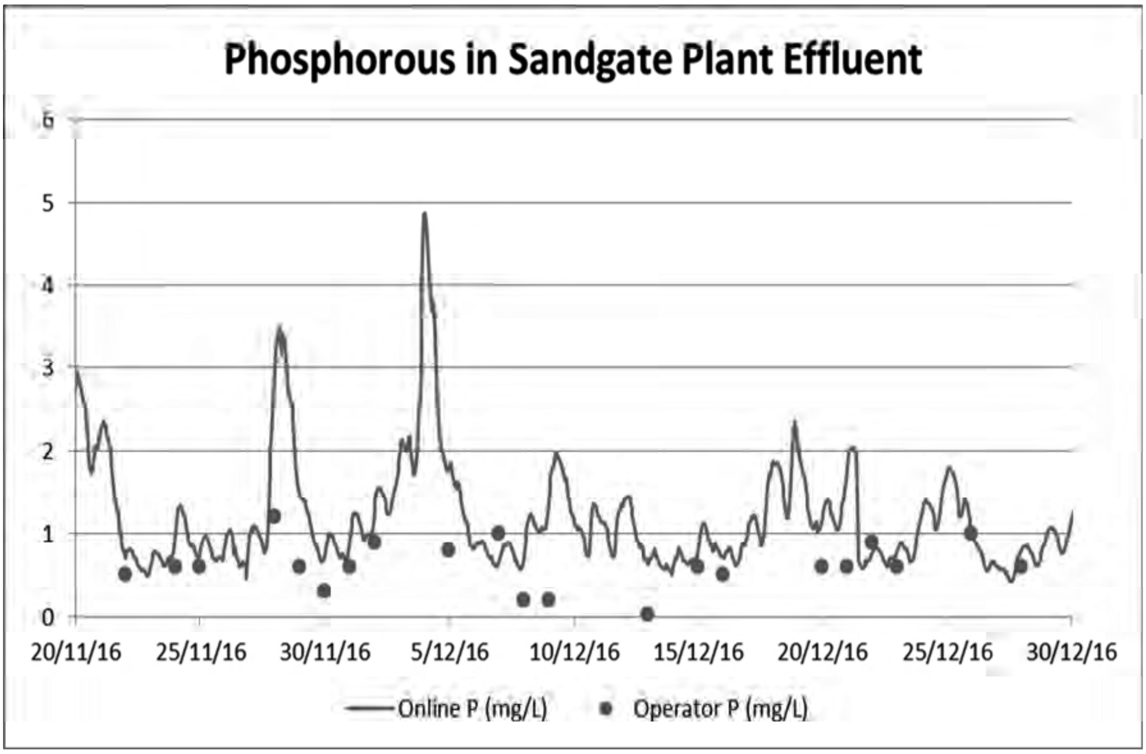


Figure 6: ***Phosphorus in the effluent***

The on line P analyser verses the operator test results are shown in Figure 6. The analyser was calibrated on 21/12/2016. After this, the variance between operator testing and on line monitoring is reduced.

2.1 The Investigation

The results prompted further investigation. The portable DO meter was taken to the field to compare the online DO probes with the portable meter. It was found that the online DO analyser was significantly higher than the portable meter (up to 1.2 mg/L higher). The difference between the online analyser and portable DO meter are shown in Figure 7.

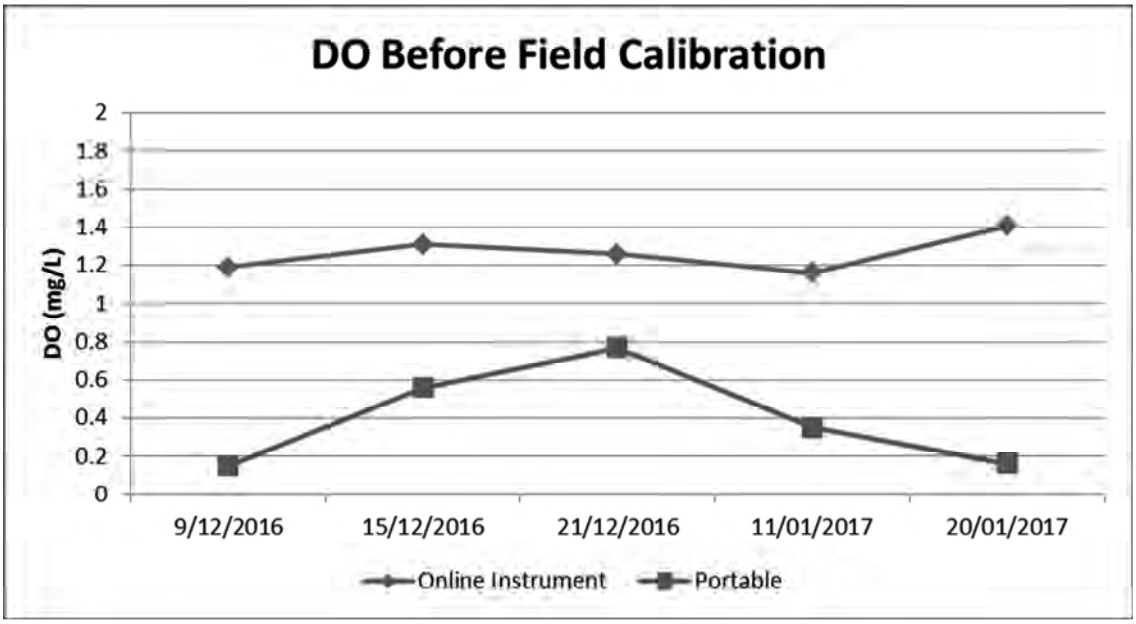


Figure 7: ***Dissolved Oxygen before field calibration***

The difference between the online DO meter and the portable DO meter was reduced after calibration (to approximately 0.2 – 0.5 mg/L) and this is shown in Figure 8.

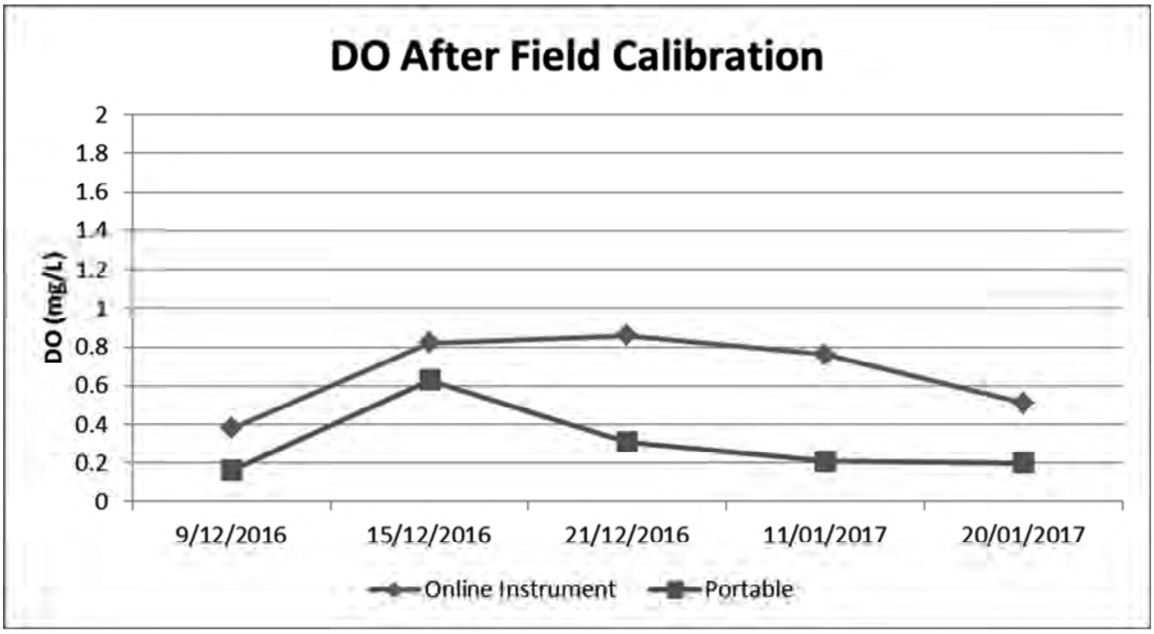


Figure 8: ***Dissolved Oxygen after field calibration***

The same test was performed on BR1 and the field transmitter and portable meter read comparably the same. This gave confidence in the results being true for BR2.

2.2 Tools for Control

A DO comparison check list was implemented to measure the differences between the field instrumentation and hand held DO meter. The hand held DO meter was draped over the ball of probes in the BR and the results recorded on the DO comparison check list to compare variances.

It was established from this check that the DO probe in BR 2 low intensity zone was reading a false high value therefore not delivering required DO to establish anoxic conditions. As a result, the settled sludge in FST 2 was turning anaerobic causing the increase in NH-4 and the release of P.

The variance between the portable DO meter and the field DO meter is shown in Table 1.

Table 1: ***Variance between the portable DO meter and the field DO meter***

	Bioreactor 2 low Intensity AIT32103B		
Date	**Transmitter**	**Hach**	**Variance**
9/12/2016	1.19	0.15	Too large
After calibration	0.38	0.16	Ok
15/12/2016	1.31	0.56	Too large
After calibration	0.82	0.63	Ok
21/12/2016	1.26	0.77	Too large
After calibration	0.86	0.31	Ok
11/01/2017	1.16	0.35	Too large
After calibration	0.76	0.21	Ok
20/01/2017	1.41	0.16	Too large
After calibration	0.51	0.2	Ok
	mg/L	mg/L	

The variance was reduced after the tilt calibration; however, it was determined that the frequency of calibration was too often to effectively monitor aeration. The operator requested field services investigate the DO probe in question. As a result the probe was replaced.

3.0 CONCLUSION

This brings about the importance of regular testing for plant performance and checking of the reliability of field instrumentation.

Whilst new technologies bring an array of online analysers and devices that form a part of the itinerary for operators to use to manage the performance of their plant, they are still just a tool. The basics of regular testing and monitoring of a plants performance should not be lost.

By having an understanding of the purpose of the different functionality of a plant, the establishment of critical set points/process parameters for a quality assurance system can be achieved.

The testing regime can then be better tailored around the functionality of the plant so swift attention can be made to conformance issues. Results outside of typical range can be brought to an operator’s attention before an STP performance is adversely affected.

4.0 RECOMMENDATIONS

Establishing critical set points for process control, and a testing /monitoring regime where results are entered into an interactive spread sheet with conditional formatting which highlights test results that are outside of the typical operational envelope.

BR 1 has an on line NH-3 analyser where as BR2 does not. This has identified the need for a NH-3 analyser for BR 2 in future works.

5.0 ACKNOWLEDGMENTS

I wish to thank the following staff from QUU, Johanna Randall, Sonja Toft, Stanley McKercher and Tony Murphy for their support and assistance in making this paper possible.

PUTTING OPERATOR SAFETY FIRST AT THE CITY OF LOGAN'S LARGEST WASTEWATER PUMP STATION

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42nd Annual WIOA
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PUTTING OPERATOR SAFETY FIRST AT THE CITY OF LOGAN'S LARGEST WASTEWATER PUMP STATION

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ABSTRACT

As the City of Logan's largest wastewater pump station, the Alfred Street pump station at Slacks Creek has continuously provided wastewater transport services to the city for more than three decades. By 2010, the pump station had become inefficient, uneconomical to operate and posed safety risks to operators. These safety risks included operating high energy switchboards in the pump machinery hall close to pump motors, and maintaining three line shaft pumps in a dry well where the pump motors and supporting beams impeded access to the confined space and where using an overhead crane was difficult.

To transform the site, Logan City Council's Logan Water Infrastructure Alliance (LoganWIA) planned, designed and delivered a $7.7 million upgrade to improve safe access for operation and maintenance, and boost the pump station's efficiency and pumping capacity. Almost $1 million was saved by reusing existing pump station components, and the project showcased innovative design and construction methods.

This paper will examine the collaborative process used to deliver a safer asset for Logan City Council operators, and the best practice techniques used to deliver the pump station upgrade.

1.0 INTRODUCTION

Completed in May 2016, the $7.7 million upgrade of the Alfred Street wastewater pump station in Slacks Creek has been a signature project for LoganWIA. The site is the main pump station for the City of Logan, servicing 200,000 equivalent persons (EP) or 84% of the Loganholme Wastewater Treatment Plant catchment. There are two separate pump stations on the site; an original pump station (SPS02) and a newer pump station (SPS69). Pump station SPS02 was constructed in 1984 as a wet well/dry well arrangement comprising three line shaft pumps with a total pumping capacity of approximately 2,400L/s. Figure 1 is a plan view of the site.

Figure 1: ***Plan of Alfred Street Pump Station site***

Asset audits and planning studies identified that the 30-year old motor control centres at SPS02 were beyond their maintainable life and posed a real safety risk to Council's operations personnel. High energy switchboards MCC1 and MCC2 were located in the pump station machinery hall close to the pump motors. Combined, these switchboards exceeded 2000A during wet weather operation.

Current electrical standards require that high energy switchboards are type tested and provide segregation to minimise the harmful effects of an arcing fault. However MCC1 and MCC2 provided inadequate segregation and had been extensively modified since manufacture. This meant that critical performance parameters were unverified either through testing records or calculations at manufacture. Personnel within the SPS02 machinery hall were potentially exposed to an electrical arcing fault hazard as a result.

While operating reliably and without major incident for over a decade, examination of MCC1 and MCC2 revealed grime collecting on terminals, unterminated cables, clutter and a corroded busbar. As such, there was a compelling case to renew the switchboards as a priority (refer to Figure 2).

<u>Figure 2:</u> ***Untidy and redundant cabling in an old MCC1 control cubicle***

Furthermore, the three existing SPS02 line shaft pumps and fittings were uneconomical to maintain and refurbish. They were identified as needing replacement from an economic standpoint and also to address safety risks inherent in maintaining line shafts (where there are many exposed moving parts).

Figure 3 shows that the existing line shaft pumps were accessed via an extensive permanent steel framework. According to AS2865-2009 Section 1.5.5, the pump station dry well is considered a confined space. The standard stipulates that enough access openings should be provided to facilitate the retrieval of injured personnel from the space. Large pump motors and supporting beams impeded the existing dry well access, leaving limited space for personnel retrieval, visual communication and maintenance activities. Furthermore, the support framework for the pump line shafts further limited overhead crane access to the well, which increased hazards associated with handling spare parts and conducting repairs. It was obvious that the steel framework would need to be removed.

Figure 3: ***Steel framework in the dry well restricted operators from removing pumps***

2.0 DISCUSSION

2.1 Approach Taken

While the need for asset renewal at Alfred Street pump station was clear, the scope was debated due to the need to keep the pump station online and to demonstrate value for money. Two studies were subsequently commissioned to define the upgrade works. One aimed to determine the optimal electrical switchboard and power supply configuration. This found that MCC1, MCC2 and MCC3 should be renewed and low voltage pumps favoured over high voltage pumps. This meant the existing building, site transformers and substation compound could be reused. A second study focused on determining the optimal pump replacement or refurbishment strategy. This study also assessed soft-starters versus VSDs and the optimum number of pumps.

The final agreed planning solution was a three pump arrangement utilising dry-mounted submersible pumps in a duty/duty-assist/standby arrangement where all pumps were controlled via VSDs. This improved operational efficiency of the pump station by enabling flow-pacing. It was also agreed that new switchboards should be located in a sealed and secure control room. Two advantages of this approach were to isolate the electrical equipment from the corrosive gas environment of the dry well and ensure operations personnel worked remotely from the main switchboard. In summary, the scope of the upgrade involved:

- Improving operator safety by removing the existing line shaft pumps and associated access platforms.
- Increasing system reliability through installation of new dry-mounted submersible pumps.
- Improving the operability of the existing odour control system.
- Eliminating electrical hazards and improving system flexibility via installation of new switchboards and variable speed drives (VSDs).

2.2 Scope Refinement

The project scope was refined using a series of tailored Design Opportunity and Risk (DOAR) workshops. LoganWIA uses DOAR workshops (at 30% and 85% design intervals) to systematically review all aspects of the design and incorporate contributions from a wide variety of stakeholders including designers, operators and constructors, as well as community, environmental and safety specialists.

Safety in design, an essential consideration for an asset's whole-of-life cycle, was also embedded into the DOAR process. Separate Hazard and Operability (HAZOP) and Construction Hazard Assessment (HAZCON) workshops were also held.

As a result of these workshops, the scope of the project was refined to avoid significant modifications to the pump station site, notably:

- Careful pump selection to eliminate the need to raise the pump station building roof.
- Use of the existing electricity supply configuration at site without upgrading the site sub-station or standby generator.
- Converting the existing odour control room to a new electrical control room rather than building a new stand-alone building (refer to Figures 4 and 5).

LoganWIA used cost-effective 3D laser surveying for all aspects of design development including confirming exact pipework lengths, acquiring difficult-to-obtain measurements (such as retracted crane hook heights) and ensuring safety in design obligations were met. The 3D survey data, and subsequent consultation with construction contractors, were also used to develop a safe work method for removing the pump line shaft framework system and associated platforms. This was via a knuckle boom spider elevated work platform.

Figure 4: ***SPS02 odour control room before the upgrade. This room was modified to become an electrical control room for the new main switchboard and VSDs***

Figure 5: ***New electrical room with switchboard (previously odour control room)***

2.3 Safety Improvements

Augmenting the electrical system

Renewing all site switchboards was an opportunity to implement industry best practice. A purpose designed, climate controlled and secure control room was built utilising existing space within the SPS02 building. This offered a significant reduction in new cable supply compared with providing a new structure elsewhere on site. Due to the high powered equipment on site, cable management and minimising cable lengths was key to reducing project costs. The building modification cost was comparable to the cost of providing a pre-fabricated control room. Installing switchboards in a purpose built control room allowed the procurement of industry-standard components. Air conditioning was used to provide temperature control for the room and a carbon filter used to scrub fresh air prior to being pumped into the room from outside. The room is under constant positive pressure with all doors closed. All mains cables were routed outside the building to avoid cable penetrations between the machinery hall and the new control room, minimising the risk of corrosive gas affecting the electrical system. Ultimately, use of a purpose designed control room has meant that operations and maintenance personnel are no longer exposed to a potential arcing fault hazard. The pump station can also be operated without an increased risk of failure due to noxious gas corrosion or high temperature stress.

Providing local control stations

In pursuing an operation philosophy that separates site personnel from electrical switchboards, local controls were installed in the machinery hall. A local control station for each pump (including the sump pumps) was designed in consultation with Council's operations personnel. A local control station for each air circuit breaker was also provided, allowing switching between regular and alternative power supply without the need to enter the control room.

Additional safety improvements

Other improvements were introduced during the pump station upgrade to enhance safety:

- An intermediate dry well gantry crane that can access all valves and fittings.
- A mobile workers' platform that provides access to the top of the pumps.
- A safety cage fall protection system around the main dry well entry ladder.
- A safety barrier around the check valve lever arms.
- Safety grills over the main pump station access opening.
- Increased open working space (refer to Figure 6) by removing steel framework.

<u>Figure 6:</u> ***Looking down into dry well (post upgrade)***

3.0 CONCLUSION

3.1 Key Outcomes

A summary of key outcomes of the Alfred Street pump station upgrade included:

- Best practice electrical design eliminated hazards to Council operations personnel working at SPS02; particularly the risk of injury to personnel from an arcing fault associated with working near high current switchboards. Initiatives included separating the electrical control room and machinery room and converting an existing odour control room to a climate controlled, sealed electrical control room (refer to Figures 4 and 5 for pre and post-upgrade comparisons). Local control stations for each pump and air circuit breakers were provided in the machinery room, eliminating the need for operations personnel to enter the electrical control room during pump station operation.
- Removal of line shaft pumps and steel framework in the dry well further reduced hazards to operators working in this confined space.
- A business cost saving of almost $1 million (about 12%) was achieved by eliminating the need for significant modifications to the pump station building roof, and avoiding the construction of a new electrical control building.
- A low corrosive atmosphere for new electrical switchboards and drives was provided by sealing the control room and running cables outside the building to minimise cable penetrations to the dry well – improving the performance and durability of the pump station.
- Increased operational reliability and flexibility has reduced risk of wastewater overflows from SPS02 to the surrounding residential area. The pump station can pump in two different directions via separate conveyance paths, and can also run at least one pump from either of two transformers fed from separate high voltage substations.

3.2 Lessons learnt

Lessons learnt from this project include:

- Through implementation of rigorous planning and design processes, brownfield pump stations such as SPS02 can be optimised cost-effectively and kept in service for many more years.
- Engagement with Council operations personnel and subcontractors in design (DOAR) and HAZCON workshops is vital to ensure that the ultimate design is safe and operable.
- 3D laser surveying is an effective tool that enables designers, constructers and operators to better understand and eliminate safety risks.
- Subcontractors can contribute valuable insights into about safe work methods for difficult activities (such as removal of the steel framework inside the SPS02 dry well).

4.0 ACKNOWLEDGEMENTS

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BIOREMEDIATION OF MUNICIPAL WASTEWATER USING FRESHWATER MACROALGAE

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BIOREMEDIATION OF MUNICIPAL WASTEWATER USING FRESHWATER MACROALGAE

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ABSTRACT

Municipal wastewater treatment plants (MWTP) discharge large quantities of treated water to the environment each day that could instead be productively used to cultivate freshwater macroalgae. We have developed the use of freshwater macroalgae as an in-line secondary / tertiary treatment process integrating the intensive cultivation of *Oedogonium intermedium* with the treated discharge water from both a 29 $ML.day^{-1}$ and a 1.6 $ML.day^{-1}$ MWTP. In both cases the cultivation of *Oedogonium* resulted in large reductions in the concentration of total nitrogen and total phosphorous. The productivity of *Oedogonium* exceeded 50 $Tonnes.Ha^{-1}.yr^{-1}$ and this biomass was of high quality with a total protein content of 23%, making it attractive as a feed for livestock. This study demonstrates that the production of macroalgae can complement the operation of conventional wastewater treatment plants by recovering residual nutrients and simultaneously creating a high-quality biomass resource that is suitable for product development.

Keywords: Algae, Bioremediation, Phosphorous, Nitrogen, Water treatment, Value-adding

1.0 INTRODUCTION

The intensive production of macroalgae, both as a tool to recover waste nutrients and as a source of sustainable biomass, has developed significant momentum in the last decade and is now viewed as a viable technology for application at commercial scales. Algae are particularly attractive as part of a transformative solution where the nutrients in wastewater can be recovered and converted into biomass such that they are available for re-use. This reduces the net export of nutrients to the environment and creates a resource that can be used to meet the ever increasing demand for food and energy. In order to implement these "closed loop" systems, and produce biomass at a scale required for it to be commoditised, there is a need to integrate the land-based production of algae with existing water and nutrient sources. Specifically, a large and consistent supply of water is fundamental as every hectare of algal production requires approximately 5 ML of water to initially fill the shallow culture ponds and ongoing daily water exchanges are required to deliver nutrients to the cultures, facilitate harvesting of the biomass and replace water lost to evaporation.

There are a range of industries that could be explored for the purpose of integration, however, the largest and most consistent source of wastewater that contains nutrients is from municipal wastewater treatment plants (MWTP). Worldwide, at least 181 km^3 of municipal wastewater is treated annually and less than 13% of this treated effluent is reused, with the majority discharged into the environment (Sato et al. 2013). Modern sewage treatment plants are very efficient at removing nutrient and biological contamination from municipal wastewater.

However, their discharge water contains a relatively high residual concentration of nitrogen (> 3 $mg.L^{-1}$) and phosphorous (> 0.5 $mg.L^{-1}$) compared to the receiving environments. In addition to these modern treatment plants there are also a large number of older treatment plants, often in regional areas that have only primary or limited secondary treatment, creating a low volume (<3 ML day^{-1}) effluent that has relatively high concentration of nitrogen (10-20 $mg.L^{-1}$) and phosphorous (5-10 $mg.L^{-1}$). When these discharge concentrations are multiplied by the total amount of water discharged daily along the Queensland coastline, it does represent a large export of nutrients to the environment that could instead be intercepted and used to cultivate freshwater macroalgae.

There are many qualities of existing discharge water from sewage treatment plants that make it an attractive water source for the intensive production of macroalgae. It is a stable source of nutrients with minimal fluctuations throughout the year and, if passed through membranes or clarified, its turbidity is low permitting high penetration of light. In addition, it is relatively clean of microbial contaminants and the microbial community present is unlikely to negatively affect the algal cultures. Consequently, the algal biomass produced in this treated water should be of a high and consistent quality, making it suitable for animal feed applications. Furthermore, the production of this algal biomass will also benefit the treatment plant itself, as the quality of the discharged water will be further improved prior to release into the environment. This could notionally extend the life of existing infrastructure as the treatment capacity of the plant could be increased thereby complying with environmental discharge limits and postponing the need for capital expenditure to upgrade the plant.

The aim of the work presented here is to demonstrate how macroalgae can be used to complement the existing process of wastewater treatment by recovering nutrients and improving the quality of the discharged water that is released into the environment, while simultaneously generating a homogenous biomass resource that is suitable for a range of product applications. In this study the freshwater macroalgae, *Oedogonium* was continuously cultivated in the treated discharge water from two municipal treatment plants; a modern 29 $ML.day^{-1}$ plant with membrane bioreactor treatment, and secondly, an older 1.6 $ML.day^{-1}$ regional treatment plant that uses conventional trickling filter technology. Specifically, we quantified the amount of nitrogen, phosphorous and metals recovered by the algae when cultured in the discharge water from each plant. At the 29 ML treatment plant we continuously cultivated the algae over a 12 month period to quantify the average monthly biomass production in this continuous culture system. Likewise we present the outcome of an initial feasibility study that uses the discharge effluent from a regional 1.6 ML treatment plant. This water was transported to James Cook University over a six week period and used as the sole source of water and nutrients to cultivate *Oedogonium*. These data provide a basis to determine the scale of algal operations and volume of products that could be supported by treatment plants.

2.0 DISCUSSION

2.1 Cleveland Bay Municipal Treatment Plant

The Cleveland Bay MWTP has a maximum treatment capacity of 29 ML and receives on average 20 ML of municipal wastewater daily. This wastewater has an average total nitrogen concentration of 55 $mg.L^{-1}$ and an average total phosphorous concentration of 8.3 $mg.L^{-1}$. After primary and secondary treatment a 10-fold reduction in the concentration of these nutrients occurs with the discharged effluent having a total nitrogen and phosphorous concentration of approximately 3-4.0 $mg.L^{-1}$ and 0.5-1 $mg.L^{-1}$ respectively.

Oedogonium was cultured using this treated discharge water in three large parabolic tanks (each 25 x 2m; surface area $50m^2$), to give a daily treatment volume of ~80 KL. *Oedogonium* was maintained in tumble culture in these tanks through a central 50 mm aeration line with air supplied by a side channel blower (Aerotech SD600). These cultures were maintained under constant flow, with each tank receiving ~27 $KL.day^{-1}$ (i.e. 100% water exchange or 1 tank volume per day) (Figure 1).

Figure 1: ***Algal cultivation system at the Cleveland Bay MWTP in Townsville***

2.2 Water Quality and Improvement after Algal Cultivation at Cleveland Bay

Over the 12 month period the incoming water to the algal treatment tanks had a mean (± SD) total nitrogen (TN) concentration of 3.2 (± 1.6) $mg.L^{-1}$, with 58.9% (1.9 ± 1.4 mg. L^{-1}) in the form of dissolved inorganic nitrogen (DIN) which is available for uptake by the algae. The majority (98%) of this DIN was in the form of nitrate-N. The average total phosphorous (TP) concentration in the incoming water was 0.9 (± 0.4) $mg.L^{-1}$, with 91.9% (0.8 ± 0.4 $mg.L^{-1}$) of in the form of reactive phosphorous (RP) which is the form available for uptake by the algae. The cultivation of algae resulted in a significant improvement in the quality of the discharged water. The concentration of DIN decreased on average by 47.1% to 1.0 (± 0.8) $mg.L^{-1}$ and TN decreased by 36.1% to 2.0 (± 1.5) $mg.L^{-1}$. Similarly, RP decreased by 68.5% to 0.3 (± 0.2) $mg.L^{-1}$ and TP decreased by 64.6% to 0.3 (± 0.2) $mg.L^{-1}$ after moving through the algal cultures. The algae also reduced the concentration of manganese by 67% (28.3 $\mu g.L^{-1}$ to 9.4 $\mu g.L^{-1}$), zinc by 42% (13.2 $\mu g.L^{-1}$ to 7.7 $\mu g.L^{-1}$), aluminium by 31% (4.9 $\mu g.L^{-1}$ to 3.4 $\mu g.L^{-1}$) and copper by 14% (2.0 $\mu g.L^{-1}$ to 1.7 $\mu g.L^{-1}$).

2.3 Annual Biomass Productivity of *Oedogonium* at Cleveland Bay

Oedogonium was cultured continuously on-site over the 12 month production period without requiring re-inoculation as this species is propagated through fragmentation. A total of 491 kg dry weight (dw) of *Oedogonium* biomass was harvested from the system between March 2015 and February 2016 at an annual average biomass production rate of 12.4 (± 3.6) g (dw).$m^{-2}.day^{-1}$. Biomass production ranged from a monthly minimum of 8.9 (± 2.6) g dw.$m^{-2}.day^{-1}$ in June to a maximum of 15.8 (± 3.7) g dw.$m^{-2}.day^{-1}$ in January (Figure 2).

In all months except June and July the average biomass productivity exceeded 10 g $dw.m^{-2}.day^{-1}$ (equivalent to 36 T $ha^{-1}.yr^{-1}$). The average monthly biomass productivity was closely related to seasonal changes in the environmental factors of temperature and light with the average summer productivity of 15.5 (± 3.9) g $dw.m^{-2}.day^{-1}$ being 64% higher than the average winter rate of 9.5 (± 2.9) g $dw.m^{-2}.day^{-1}$. The average biomass productivity during autumn and spring was similar at 12.6 (± 3.4) g $dw.m^{-2}.day^{-1}$ and 12.1 (± 3.1) g $dw.m^{-2}.day^{-1}$ respectively.

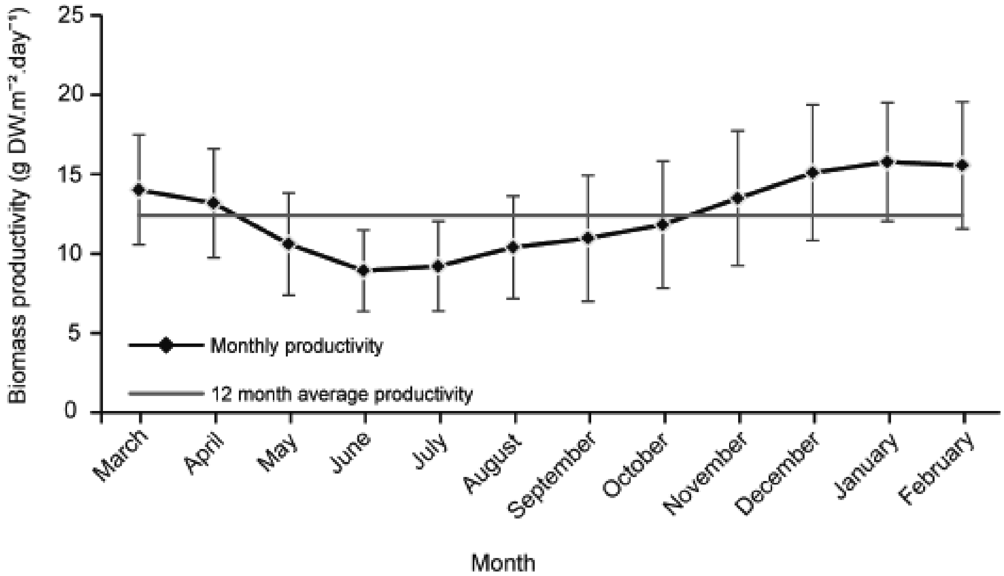

Figure 2: ***Monthly biomass productivity of Oedogonium over a 12 month period when cultured in the treated discharge water from the Cleveland Bay municipal wastewater treatment plant. The data is the average (± SD) of each harvest made during each of the 12 months***

2.4 Biochemical Composition of the *Oedogonium* Biomass Cultured at Cleveland Bay

The biomass had a 12 month average internal nitrogen content of 5.0 (± 0.1) % (range 4.5-5.4%), phosphorus content of 0.9 (± 0.1) % (range 0.8-1.1%) and carbon content of 42.4 (± 0.2) % (range 41.7-43.6%). In total 24.4 kg of nitrogen, 4.8 kg phosphorus and 208.1 kg of carbon were recovered by the *Oedogonium* biomass over the 12 month period. This equates to an annual uptake rate of 1.6 T of nitrogen, 0.3 T phosphorous and 13.9 T carbon per hectare. The four most abundant mineral elements in the *Oedogonium* biomass were potassium (31.1 $g.kg^{-1}$), phosphorous (9.3 $g.kg^{-1}$), calcium (5.3 $g.kg^{-1}$) and magnesium (5.0 $g.kg^{-1}$). Of the remaining elements manganese, iron, zinc, strontium and aluminium had concentrations ranging between 98.1 and 415.7 $mg.kg^{-1}$. The remaining elements (Ba, Cu, As, Se, Pb, Ni, Cr. Mo, Co, V) were present at very low concentrations. Importantly, none of the potentially toxic elements, such as As, Pb, Hg or Cd, were found in concentrations high enough to be problematic for algal growth or impose restrictions on product development from the biomass. The total protein content (sum of amino acid) of the *Oedogonium* biomass was 23.1 (± 0.1) $g.100g^{-1}$, with 10.0 (± 0.1) $g.100g^{-1}$ of essential amino acids. The two essential amino acids methionine and lysine, which are important for animal nutrition, had a mean concentration of 0.4 (± 0.1) $g.100g^{-1}$ and 1.6 (± 0.01) $g.100g^{-1}$ respectively. The total lipid content of the biomass was 10.4 (± 0.1) $g.100g^{-1}$ and a total fatty acid content of 6.4 (± 0.1) $g.100g^{-1}$. The fatty acid composition was dominated by polyunsaturated fatty acids which comprised 71.2 (± 0.1) % of the total fatty acid pool, while saturated fatty acids and monounsaturated fatty acids accounted for 22.7 (± 0.2) % and 6.1 (± 0.2) % respectively.

The biomass had a total dietary fibre content of 34.5 (± 0.2) $g.100g^{-1}$ with the majority (94.8%) of this fibre insoluble.

2.5 Burdekin Treatment Plant

The Burdekin Shire Council (BSC) municipal treatment plant discharges on average 1.6 ML of secondary treated effluent per day. This discharge has a total nitrogen concentration of 11.6 (range 8.7-22.1) $mg.L^{-1}$, total phosphorous concentration of 8.8 (range 7.4-10.7) $mg.L^{-1}$ and total suspended solids of 5.7 (range 0-20) $mg.L^{-1}$. To demonstrate that this water is of a suitable quality to cultivate *Oedogonium* we transported the discharge water to James Cook University over a six week period and quantified the biomass production and improvement in water quality. In these trials the algae was cultured in seven day cycles in 20L buckets filled entirely with the discharge water from the Burdekin MWTP. These cultures were compared to control cultures maintained in dechlorinated tap water with synthetic nutrients added.

2.6 Algal Productivity and Improvement after Algal Cultivation in Burdekin Water

Over the six consecutive weeks *Oedogonium* survived and grew well in the discharge effluent from the BSC municipal treatment plant. The six week average (± SD) productivity of the *Oedogonium* in the BSC discharge effluent was 19.1 (± 1.5) g dw m^{-2} day^{-1} (equivalent to 70 T $ha^{-1}.yr^{-1}$), which was 16% higher than the control treatment (15.9 ± 1.5 g dw m^{-2} day^{-1}) which was dechlorinated tap water with synthetic nutrients. Culturing *Oedogonium* in the discharge water significantly improved the quality of the water and over the six week period the TN was reduced by 84.1 % (± 2.8) % from 9.9 (± 1.1) to 1.5 (± 0.2) mg L^{-1} and TP was reduced by 61.1 (± 5.9) % from 7.7 (± 1.1) mg L^{-1} to 3.0 (± 0.8) mg L^{-1}. DIN was reduced by 99.8% from an average of 8.3 (± 1.0) mg L^{-1} to trace quantities of 0.02 (± 0.01) mg L^{-1}. Similarly, the concentration of RP in the 100% BSC treatment was reduced by 65.2 (± 7.5) % from 7.2 (± 0.9) mg. L^{-1} to 2.5 (± 0.8) mg. L^{-1}.

2.7 Implementing the Scaled Production of *Oedogonium*

Oedogonium cultured in the treated discharge water from two MWTP recovered residual nutrients, yielded consistent production relative to environmental conditions and delivered a biomass resource with consistent biochemical composition. Importantly this biomass had a high concentration of essential amino acids and polyunsaturated fatty acids with potential end uses as an animal feed. An important outcome of this study was that *Oedogonium* was continuously propagated in situ over a 12 month period without the need for re-inoculation or strict culture management. This work is the first to determine the biomass productivity of *Oedogonium*, or any freshwater macroalgae, over a longer timeframe (12 months) and provides a monthly average rate of biomass production relevant to large-scale cultivation systems. As an example of the potential scale of operations, if all 20 ML of the treated discharge water from Cleveland Bay MWTP was used to culture *Oedogonium* with a 100% water exchange rate, then a production area of 4 hectares (with a depth of 0.5 m) could be supported which would produce an annual crop of 181 T dw.

A key finding of this study was that the cultivation of *Oedogonium* can complement the existing operations of both energy intensive modern, and older regional sewage treatment plants. The discharge effluent from both the Cleveland Bay and Burdekin treatment plants were of a suitable quality and do not require any additional pre-treatment prior to use as a medium for algal cultivation.

Integrating the cultivation of *Oedogonium* into existing treatment plants can create a mechanism through which value can be added to conventional MWTP. Algae can add value to existing MWTP through several complementary mechanisms, which include decreasing the amount of nutrients exported to the environment, savings from reduced capital expenditure associated with minor upgrades in the capacity of the plant and by providing a source of direct revenue through the development of products derived from the cultured biomass. From an environmental perspective, the prevention of nutrient export is an important aspect of maintaining healthy ecosystems and is particularly important for regions, such as Australia's north eastern coastline, where wastewater treatment plants discharge into sensitive marine environments. In the case of Cleveland Bay MWTP, the cultivation of *Oedogonium* as a tertiary polishing component of water treatment would reduce the export of nitrogen by between 8.4-10.5 $T.yr^{-1}$ and phosphorous by 3.8-4.2 $T.yr^{-1}$. Importantly, if there are load-based regulations for the export of nitrogen and phosphorous, then the algae can create value for the plant through cost savings, where instead of upgrading the treatment plant itself the excess nutrients can be recovered through the modular expansion of the area under algal cultivation. *Oedogonium*, can potentially extend the life of the existing infrastructure and reduce the frequency of capital expenditure associated with upgrading the capacity of the treatment plant to both accommodate population growth and to comply with environmental discharge limits.

3.0 CONCLUSION

Integrating the production of *Oedogonium* with MWTPs provides a unique mechanism to recover and exploit the inherent value held within municipal wastewater, converting the residual waste nutrients into high-quality algal biomass that can be used for a range of bioproducts. However, the critical step in developing a scale process was to firstly confirm that *Oedogonium* can be reliably cultivated over the long term in treated municipal wastewater and that the biochemistry and therefore product streams of this biomass are stable. We have now demonstrated all of these. Currently municipal wastewater, and the nutrients it contains, are viewed predominantly as a liability. Instead, if this discharged water and nutrients were passed through an intensive algal production system a significant quantity of residual nutrients can be recovered, complementing and adding value to conventional wastewater treatment plants. In the longer term, conventional wastewater treatments plants could be repurposed and developed as integrated waste to product plants where a range of algal-based bio-products can be developed to meet existing and future demands for proteins, fibre and phosphorous.

4.0 ACKNOWLEDGEMENTS

This research is part of the MBD Energy Research and Development Program for Bioremediation and Bio-products. The project was supported by the Australian Government through the Australian Renewable Energy Agency (ARENA 002369). The authors would like to thank Mr Dean Davis and the staff from both the Cleveland Bay and Ayr MWTP that facilitated this project.

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USING ONLINE NUTRIENT ANALYSERS TO OPTIMISE BOTH CHEMICAL DOSING AND AERATION SYSTEMS

Paper Presented by:

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Cairns Regional Council

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USING ONLINE NUTRIENT ANALYSERS TO OPTIMISE BOTH CHEMICAL DOSING AND AERATION SYSTEMS

Mark Gwynne, *Treatment Coordinator,* Cairns Regional Council
Aron Smith, *Senior Operator Leading Hand,* Cairns Regional Council

ABSTRACT

The installation of an online Nutrient analyser in the Bioreactor at Edmonton WWTP has allowed the staff to optimise both the Alum dosing and Aeration systems and save significant amounts of chemicals, reduce electricity consumption while still maintaining plant process.

To date the savings realised in chemicals are over $12,000 per year. Electricity savings are over $17,000 per year.

Further efficiencies were obtained from a reduction in onsite lab testing. Operators have reported that they saved approximately 2 hours per week in the laboratory which offsets the time spent on the analyser. In addition, laboratory supply costs have also reduced.

Having the analyser hooked up to the SCADA system has also allowed for a more improved monitoring system to detect process upsets. A sample if analysed every 20 minutes and each nutrient channel has high level alarms along with normal trends to allow for great process control and an earlier warning system of a treatment issue.

1.0 INTRODUCTION

The Edmonton wastewater treatment plant is a relatively typical Activated Sludge system with an Oxidation Ditch and Clarifier. The average dry weather flow is approximately 4.5 ML/day. The catchment contains a little over 20,000 people. In 2009, the plant received an upgrade to cover a new license that contained a Total Phosphorus limit of 1mg/L 50^{th} percentile and 3 mg/L maximum. The average Phosphorus coming into the plant is 7.3 mg/L.

There is no facility for Biological Phosphorus removal. The plant upgrade provided chemical dosing for Phosphorus removal using Alum.

Previous optimisation efforts for Alum dosing saved the council significant amounts of money. However, with no online monitoring of nutrients, the operators could do little to reduce dosing set points any further. After-hours testing was difficult to get accurate results, as there was no site attendance throughout the night, as well as no site attendance on Saturday, and just two staff for 3 hours on Sunday.

In late 2014, an online Nutrient analyser was installed with commissioning completed in 2015. This instrument now gave the operator real time data of Ammonia, Nitrate and Phosphate every 20 minutes from the Bioreactor.

Although optimisation work had already made significant savings, the 2014/15 annual Alum cost was over $75,000.

The plant’s total Phosphorus results against the license for the financial year, prior to the online analyser optimisation program are shown in Figure 1.

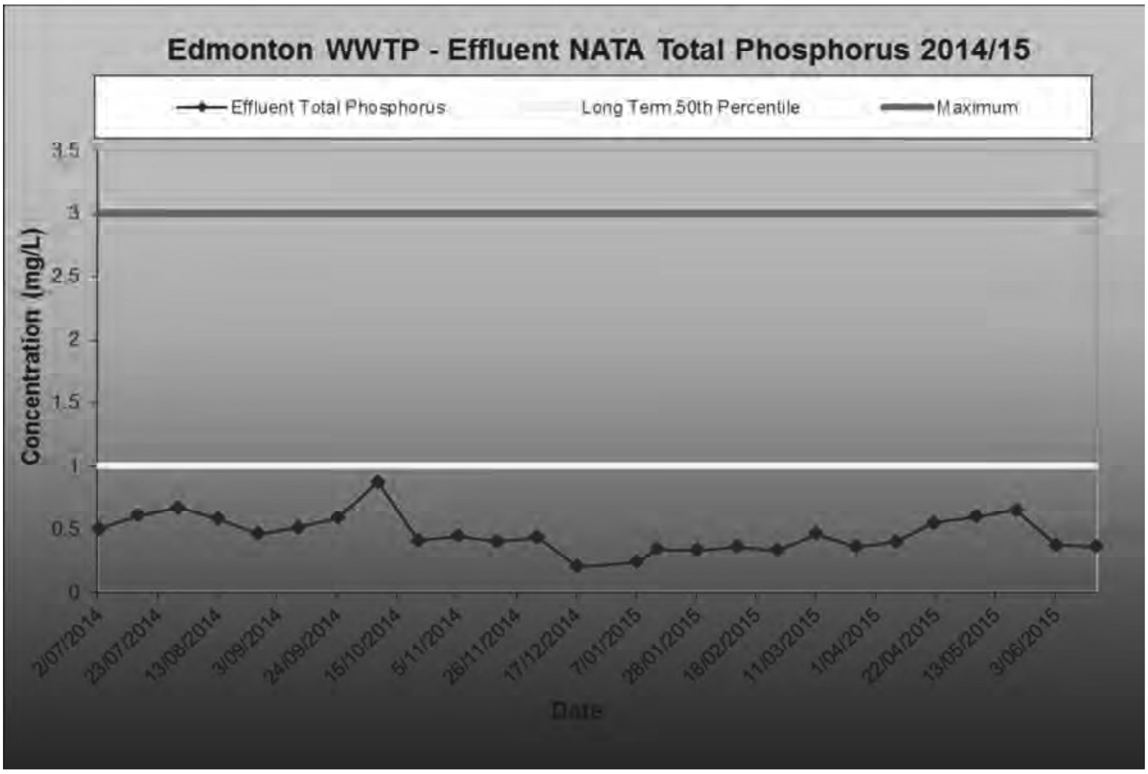


Figure 1: ***Edmonton WWTP Effluent Total Phosphorus***

2.0 DISCUSSION

After a detailed optimisation program for Alum using an automatic sampler on the effluent and grab samples on the Bioreactor, the results still indicated low Total Phosphorus on the effluent. However, the risk of any further reductions to the dose rate was too high without a more detailed testing program. A similar issue applied to aeration. For both systems it was identified that further optimisation was possible but we needed more data.

For this to occur, after hours sampling would have been required including weekends. The normal roster meant no effluent was tested on a Friday night/Saturday so any dose rate changes in this period resulted in our greatest risk in case something occurred. Minimal onsite attendance on Sunday morning was also an area of higher risk. Catchment flows indicated a different flow pattern on weekends, so weekend dosing was set higher to accommodate both increased flows and decreased staffing attendance. The plant SCADA system even allowed for this in various process units. Alum dosing and aeration had separate set points to cover both Saturday and Sunday as individual days.

More frequent monitoring of the Bioreactor was only practical using an online sampler to perform testing 24/7.

2.1 Alum Savings

A program was set up to start reducing the dose rates based on the online Bioreactor dissolved Phosphorous readings along with the onsite effluent Dissolved Phosphorus and NATA Total Phosphorous tests.

The online Bioreactor results were the primary driver and the other results were used as a cross check along with regular direct calibration checks.

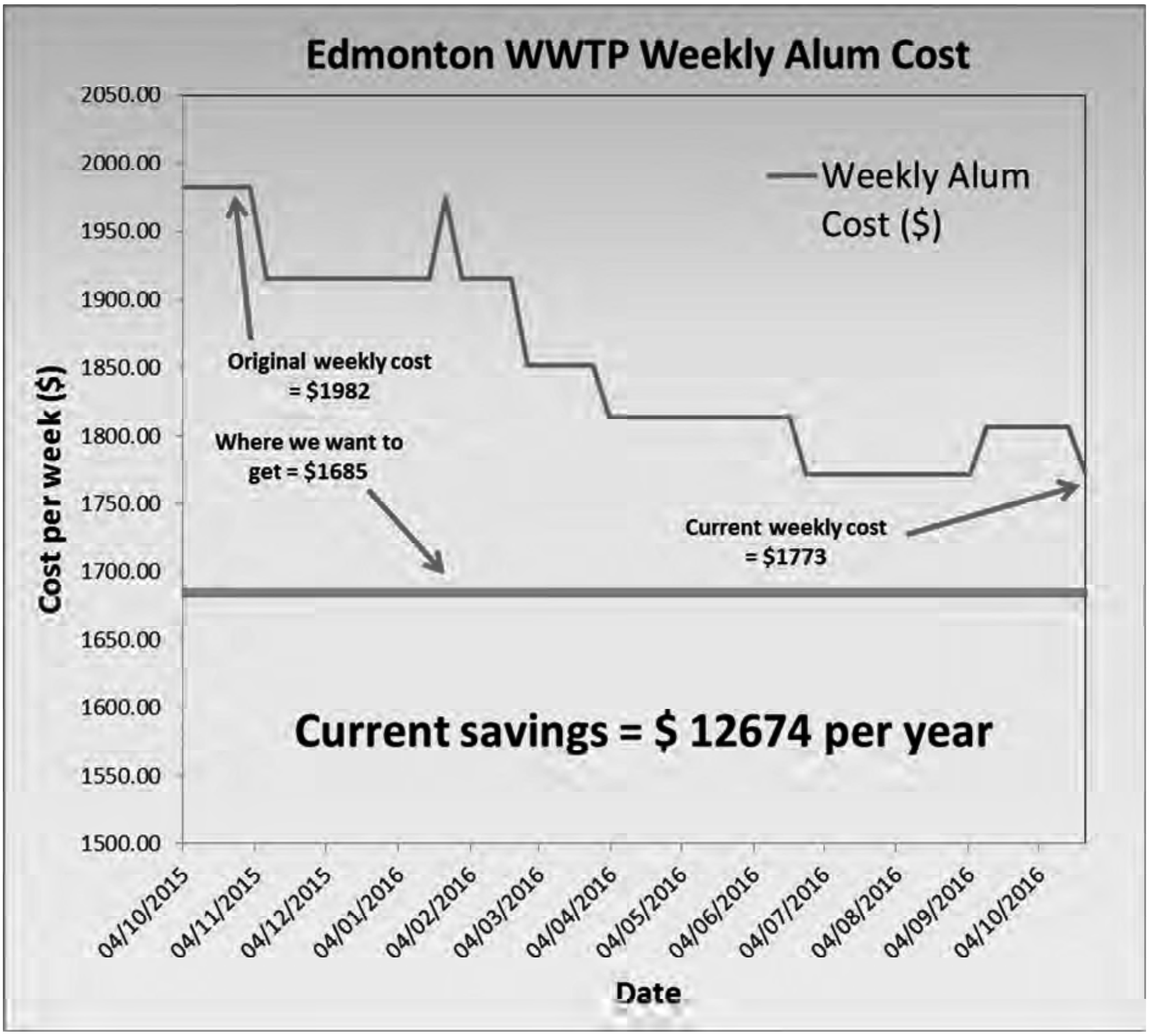


Figure 2: *Edmonton WWTP Alum dosing savings*

Note the target figure is simply a 15% reduction.

Table 1: ***Summary of savings made to Alum dosing at Edmonton WWTP 12 months before vs 12 months after***

	Alum usage (L)	Total Cost ($)
Before	261,300	$103,057
After	229,164	$90,382
Total savings	32,136	**$12,674**
% savings	12.3	**12.3**

2.2 Power Savings

Initially it was predicted some additional savings to power could be made via optimisation of the aeration system.

Edmonton WWTP operates under the Ergon energy Tariff 45 account as shown in Figure 2

Tariff 45 Demand Medium

The demand threshold under this tariff is 120kW.

	GST incl. from 1 July 2015	GST incl. from 1 July 2016
Demand charge - dollars per kW of chargeable demand per month	34.780	34.812
All usage - cents per kWh	11.529	13.632
Supply charge - cents per day	17,991.234	18,328.384

Figure 3: ***Ergon Energy Electricity Tariff pricing for Edmonton WWTP***

Peak demand is calculated by multiplying the maximum power used by the plant at any one time of the month, by the peak charge.

Savings to general use and perhaps a small reduction in the peak demand were thought possible although both were hard to predict until online monitoring results became available.

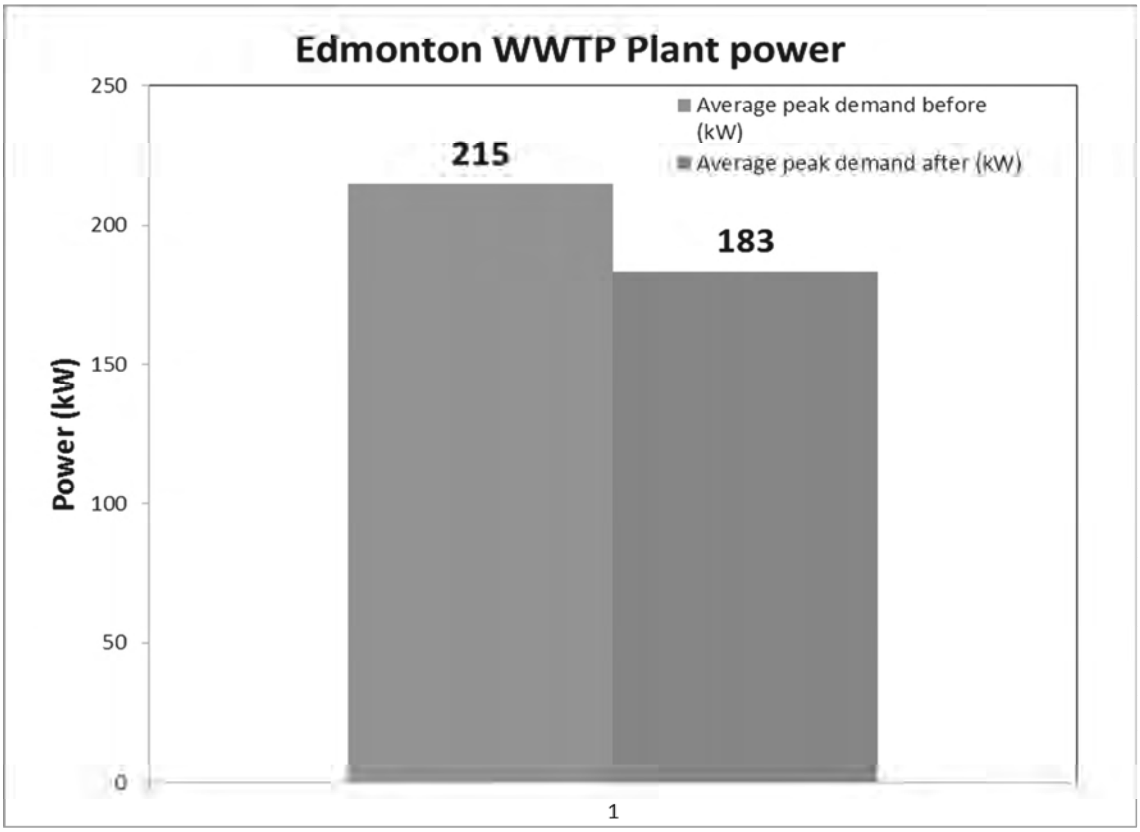
However, not long after commissioning, staff noticed a couple of items. One was that the aeration could be fined tuned a little in the morning. The main change was delaying the time the aerators sped up in the morning to match the incoming load. The second item was the time the belt press ran. When the press ran it drew almost 30kW of power. With the morning aeration time change, we were able to run the belt press outside the times the aerators worked at their hardest. The result was the staff were able to cut down on the peak demand far more than initially anticipated.

In the 12 months since the aeration and belt press run time changes, the plant achieved a saving of over $17,000. The table below shows the majority of this saving was associated with the reduction of the peak demand usage.

A monthly average reduction of 31 kW in peak demand resulted in an almost 40% reduction in the annual peak charging.

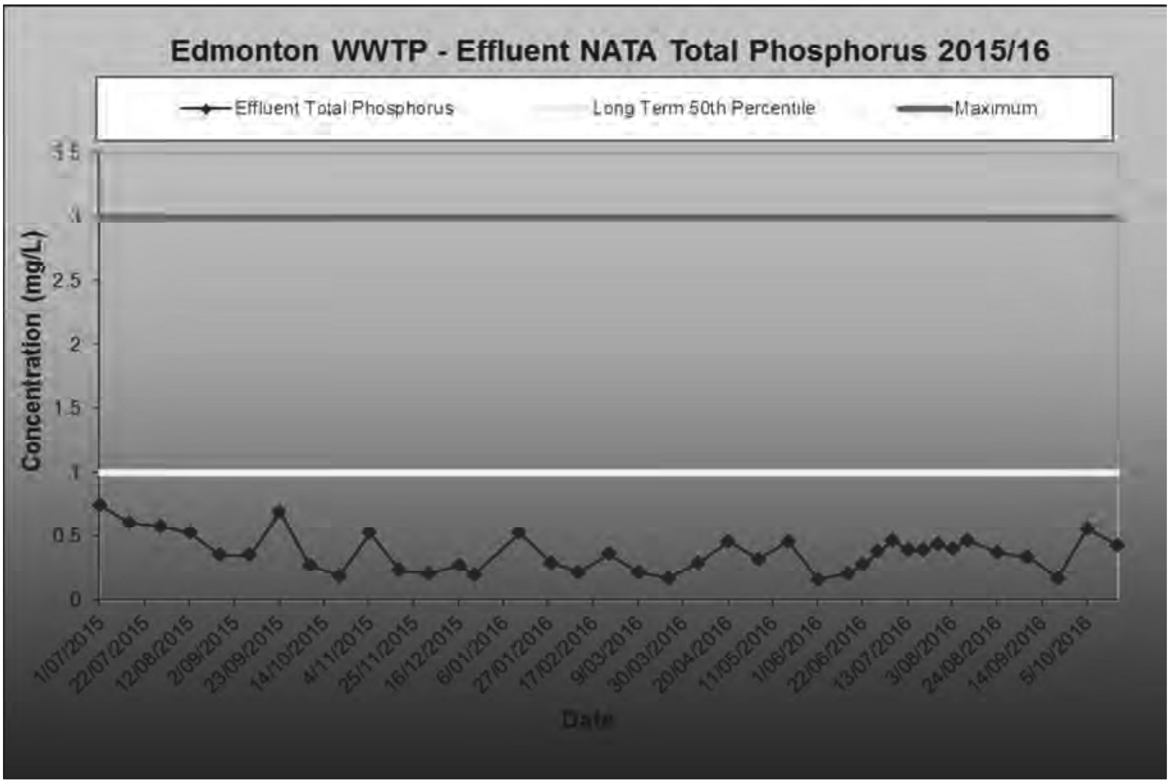
Table 2: ***Summary of savings made to power 12 months before vs 12 months after***

	Total Power (kWh)	**Average Peak Demand (kW)**	**Peak Demand cost ($)**	**Total Cost ($)**
Before	1,073,521	215	$39,875	$204,259
After	1,014,720	183	$24,105	$186,851
Total savings	58,801	367	**$15,770**	**$17,408**
% savings	5.5	15.4	**39.5**	**8.5**



<u>Figure 4:</u> ***Ergon Energy Electricity peak demand figures for Edmonton WWTP***

The plant's total Phosphorus results against the license for the optimisation period covered in the report are shown in Figure 4.



<u>Figure 5:</u> ***Edmonton WWTP Total Phosphorus results***

2.3 Other Costs and Savings

Some maintenance work has been done in the period after commissioning. The cost was around $3,465. Approximately $2,550 was part of a warranty issue. Some of the other costs involved work due to temperature issue relating to the installation of the instrument that affected the initial calibration.

Annual reagent costs are in the order of $1,600 per year.

From a staffing perspective, the 2 hours per week spent on the instrument is offset by the 2 hours per week saved by doing onsite laboratory tests. Onsite laboratory reagents are also used less so there is a saving here (unable to calculate accurately at this point).

Weekend overtime has also been looked at across all of the treatment plants. An online nutrient analyser has significantly reduced our risk at Edmonton treatment plant where the staff no longer visit the site on Sunday. This has resulted in additional savings at a reduced risk compared to other plants that do not have the analyser. This has resulted in a further savings of around $9,000 per year.

Commissioning staff time was not booked to the project specifically and was a once off.

3.0 CONCLUSION

The installation of an online nutrient analyser at the Edmonton Waste Water Treatment Plant resulted in around $25,000 worth of savings in the 12 month periods after work had commenced in the areas of Alum dosing and Aeration control. The Capital cost of the work was $100,318. Taking into account some commissioning costs, it is estimated the instrument will have paid for itself in around 5 years.

Monitoring nutrients online in this manner not only offers costs savings, but better process control and ability to meet the plants license. It can be alarmed to alert the operators to an issue at the plant that may otherwise be undetected until it is too late.

4.0 ACKNOWLEDGEMENTS

Cairns Regional Council would like to thank the staff at Edmonton Waste Water Treatment Plant for the efforts in this project. A special thanks to Graham Moroney who took ownership of the analyser and was instrumental in achieving the outcomes observed to date. Also thanks to Aron Smith and Geoff Parkes who were a great help with this project.

SMALL TOWN WATER PLANT SET TO GO AUTOMATIC

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42nd Annual WIOA
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SMALL TOWN WATER PLANT SET TO GO AUTOMATIC

Kristy Walker, *Assistant Treatment Plant Operator*, Banana Shire Council

ABSTRACT

Built in 1970, the Moura Water Treatment Plant (WTP) has had minor upgrades undertaken over the years, but with some of the outdated plant and equipment still in operation, it was becoming a major cost to Banana Shire Council for repairs and upkeep.

Applying for a government subsidy was the first step in the plant upgrade process and after obtaining approval 5 years ago, a 10 year plan was put in place, allowing the upgrades to be completed over 3 stages.

1.0 INTRODUCTION

The original Moura WTP consisted of a flash mixer, a high sludge blanket clarifier, a low lying sludge clarifier, two round media filters, two square media filters and disinfection using chlorine gas. The sludge from the clarifier is dropped manually to a sludge lagoon. The backwash water from the filters also goes to the lagoon.

Much of this equipment had become outdated and it was becoming increasingly difficult to obtain spare parts to keep things running. The maximum capacity of the low sludge clarifier was only 50L/s and the high sludge blanket clarifier 25L/s, once the high blanket sludge clarifier and two square filters taken offline at the start of the project. There are currently two raw water pumps supplying water to the plant from the Dawson River at a total capacity of 55L/s.

The treatment plant capacity necessary to meet projected future demand is 100L/s. To meet the future demand, Council has plans to install three new raw water pumps and to construct a new raw water supply main capable of handling a 100L/s flow rate.

Figure 1: ***Old Moura WTP including the clarifier and two of the round media filters***

As a result, the new plant was designed and constructed. It consists of two lamella clarifiers, both with a flash mixer, three desludging valves each, one weir between them, and two sand filters per clarifier (totalling four). When being operated at full capacity, the lamella clarifiers can supply 50L\s.

To obtain the required output of 100L/s, both plants must still be operated. Integrating a new plant with an old plant was no easy task, many things can happen and they have. This paper will discuss the backwashing, desludging, chemical dosing and the overall performance of the two plants working as one.

Figure 2: ***The new Moura WTP***

2.0 DESLUDGING

To desludge the old clarifier, there is a 50mm pipe and valve which opens on a timer that the operators set, depending on the amount of sludge being settled out of the raw water coming into the plant. The valve can also be opened manually. The new clarifiers are the same, but the sludge level can be dropped much faster, as the pipe work is 100mm and there are 3 draw out points on each clarifier.

Figure 3: ***The new Moura WTP clarifiers***

Some of the sludge outlet solenoids on the new clarifier would open and not close, and some would close and not open, which caused problems. The main issue is a build-up of sludge levels in the clarifier and then having floc carry over into the filters causing high turbidity.

Figure 4: ***The new desludging valves and solenoid***

As this happened over the Christmas period, the quick fix to this problem was to shut down the whole new side of the plant until the contractors could return and rectify the issue. Once all the solenoids were replaced the plant was started up again, only to have a new problem arise. It was thought that fixing the solenoids would solve the desludging issue.

As a result of the how the process was designed, there were problems with sludge blockages in the pipework from the flash mixer to the clarifier. This caused the flash mixer to flow out the overflow bell and also reduced the amount of flow going into the clarifier. This caused additional problems with carry over floc going through to the filters.

To solve this issue, it was believed that the manhole covers need to be removed and inside the pipes blasted with high pressure water to unclog them. Once this cleaning is completed, the desludging should operate automatically and the problem should be solved. To date, the contractors have not returned to site to undertake this work. With the plant not yet fully commissioned, the only solution was to shut down the new plant again and solely run on our old clarifier and filters.

At full capacity, the existing pumps deliver 55L\s which exceeds the capacity of the old clarifier. After variable speed drives were installed on the raw water pumps, through the CITEC program, the flow rate has been reduced to 50L\s which has stopped the clarifier from over flowing.

The amount of colour coming into the plant from the raw water, determines how long and how often we need to desludge.

Table 1: ***Desludging run times***

	Desludge For	Desludge Every	Wait Time Between Valves	Time Until Next Valve
Low Sludge Clarifier	60 Secs	6 Min	0	0
Lamella Clarifier	15 Secs	110 Min	10 Secs	10 Secs

3.0 BACKWASHING

With the old plant, backwashing is undertaken manually by the operators daily, sometimes twice each day, depending on the colour & turbidity of the raw water and how many hours the plant has operated.

Figure 5: ***One of the old round filters during a backwash***

By comparison, backwashing of the new plant was designed to be controlled by turbidity but this has been changed to hours run by the operators with turbidity used as a shutdown.

Table 2: ***New system backwashing sequence***

Backwashing Time Start Filter 5&6	Backwash Run Time Filter 5&6	Backwashing Time Start Filter 7&8	Backwash Run Time Filter 7&8
18 hours	6mins	18 hours	6mins

The turbidity and colour of the raw water in the Dawson River is extremely variable and if the colour of the River changes dramatically or quickly, the plant would have gone into permanent backwash mode, had the changes not been made.

New control panels have been installed for the backwashing system on the old plant, but due to issues with the age of the valves, the limit switches (that are now bypassed) were not able to keep the valve in position properly. This part of the upgrade is still yet to go automatic.

In contrast, the new system is fully automatic but with this, also came issues. Even though raw flow to the plant had been isolated, the CITEC controls for this plant had not been turned off. A backwash valve had opened when it wasn't supposed to, allowing a high turbidity flow to register on the on-line turbidity analyser. This caused the new plant to go into a backwash sequence, from which it would not have been able to stop.

The backwash cycle is designed so that when high turbidity is recognised for 1 hour, the plant goes into automatic shutdown and this triggers a backwash sequence. Once this backwash process is complete, there is an allowance of 1 hour for the filter ripening period. With online turbidity analysers and daily testing, having the plant go into backwash or be shut down on turbidity should rarely happen.

Another issue still to be resolved involves the media in the 4 new filters. The media currently being utilised is much finer in grade and is slipping down through the filter nozzles and clogging them. Rectifying the media will ensure we have an even distribution of water and air when backwashing.

4.0 OTHER WORKS

In addition to the new plant and equipment, there have been a number of other improvements made. We have now gone from having a flash mixer, to a 250,000 litre raw water tank.

Coagulant and pH correction
New dosing skids have been installed for pH correction (Caustic Soda). We still dose with Ultron 44560 (coagulant) & Magna Floc (lt25) for our flocculation process.

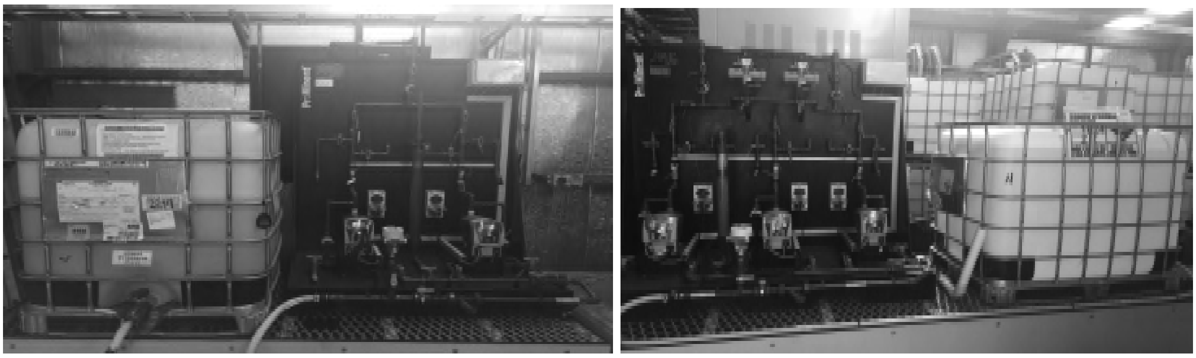

Figure 6: ***Coagulant and pH correction dosing skids***

Potassium Permanganate Dosing
A potassium permanganate dosing system has been installed to assist with iron and manganese removal in the raw water. This system will only be used when necessary.

Figure 7: ***Potassium Permanganate dosing system***

Powdered Activated Carbon
In case of algal blooms on the Dawson River or when taste and odour compounds are present, a Powdered Activated Carbon (PAC) dosing system has been installed. As with the Potassium Permanganate system, the PAC system will only be used when necessary.

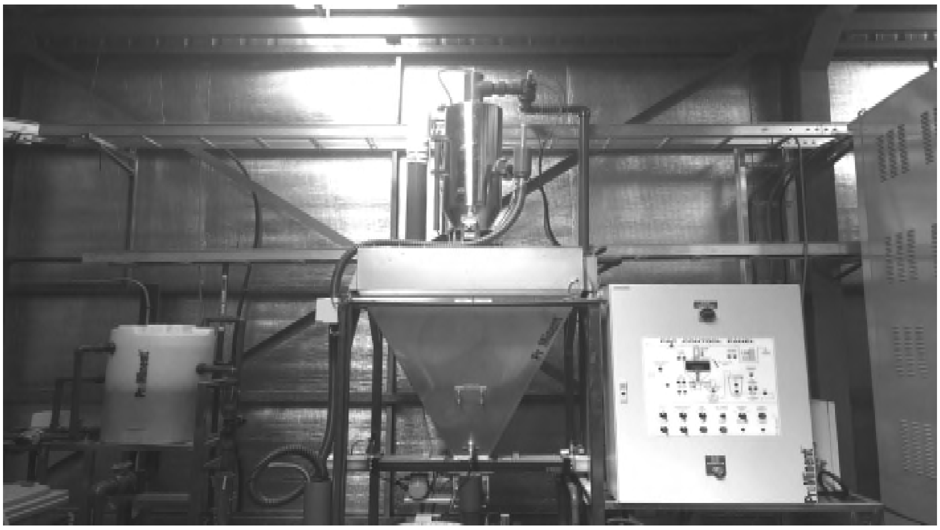

Figure 8: ***Powdered Activated Carbon system***

4.0 CONCLUSION

Having worked with Banana Shire Council for just over 2 years, I have been involved with this project from the beginning.

The work has been completed in stages and there are still a number of commissioning issues to be resolved with the new plant.

The final stage of the project will involve the installation of three new raw water pumps at the Dawson River offtake along with 5km of new raw water supply main, consisting of 400mm OD poly pipe. When the new pumps, both the new lamella clarifiers and our existing low sludge clarifier are able to be operated together, the plant will have an output of 100 L/s meeting our predicted future demand.

The new plant, along with the addition of the Potassium Permanganate and PAC systems will significantly improve the overall water quality supplied to Moura.

As the new dosing system is a lot more automated, it will be able to react to changes in raw water pump flows, or if one of our dosing pumps fails the second one will kick in. If both fail, the plant will automatically shut down where before the upgrade this was not the case. This is just one example of the many positives that the new upgrade has brought to the Moura Water Treatment Plant. It will be exciting to continue to supply water quality of a high standard to the residents of Moura.

5.0 ACKNOWLEDGEMENTS

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Banana Shire Council for giving me the opportunity to present my paper.

WETALLA BIOREACTOR MIXER BASE PLATE REMOVAL TOOL

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7 & 8 June, 2017

WETALLA BIOREACTOR MIXER BASE PLATE REMOVAL TOOL

Marcus Boyd, *Senior Treatment Officer*, Toowoomba Regional Council

1.0 INTRODUCTION

The Wetalla Waste Water Reclamation Facility is owned and operated by the Toowoomba Regional Council. The facility is a Biological Nutrient Removal (BNR) process treating an Average Dry Weather Flow (ADWF) of 36 ML/D. The plant consists of two identical bioreactors operating in parallel. Bioreactor 1 was commissioned in 1995 while Bioreactor 2 was commissioned in 2006.

During the biannual maintenance program, maintenance staff noticed significant wear on the mixer mast locating pin (Figures 1& 2).

Figure 1: ***Mixer mast***

Figure 2: ***Repaired locating pin***

Each mast locates into a base plate fixed to the bioreactor floor (Figure 3 & 4).

Figure 3: ***Worn baseplate***

Figure 4: ***Worn baseplate missing locating cone***

There are 14 mixers in the bioreactor and all locating pins had evidence of similar wear. Each pin was repaired to original specifications in the workshop. However, given the extent of the wear observed on these pins, there were serious concerns expressed regarding the integrity of the base plates themselves.

An in-situ inspection was not possible as the bioreactor could not be taken offline without causing problems to the process. To work around this restriction, a commercial diving contractor was engaged to remove a single base plate for inspection. The first attempt to remove a base plate failed as it had locked onto the anchor bolts, making removal impossible without any specialised tooling.

2.0 BACKGROUND

Biannually the mixer masts are removed for inspection, fitting of sacrificial anodes, and an application of a protective coating for corrosion control. During their operational life, the base plates had worn to a point where the masts were required to be rotated 90° to allow removal. This 90° rotation was necessary to free the mast from the groove that had worn into the base plate. Unable to inspect the base plates, concerns were raised as to how long the base plate could remain in service before a critical failure occurred.

Given the requirement for effective mixing in the bioreactor, a critical failure was not an option. Initial investigations took place into taking the bioreactor offline and replacing the base plates. This process proved to be expensive ($130,000+), while potentially compromising final effluent discharge, licence compliance requirements as well as the unnecessary disruption involved with taking a bioreactor offline.

The wear on the base plates and locating pins was a result of constant rubbing caused by the normal operation of the mixers, combined with the abrasive nature of the fine grit within the wastewater. This rubbing had been continuous for the past 22 years. The mast locating pins were repaired when required, while the base plates continued to wear. Approximately 18 months ago, it became apparent the base plates would need to be replaced.

3.0 THE SOLUTION

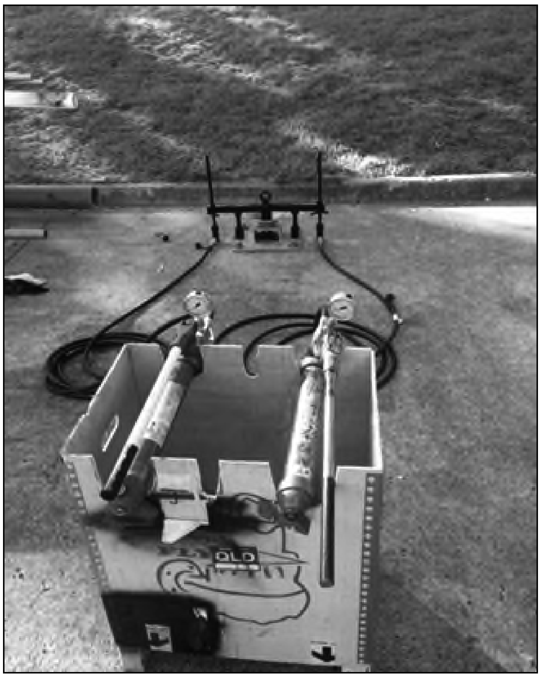
While the investigation was underway to take the bioreactor offline, the 14 replacement baseplates were procured from the original supplier. The time to order and fabricate the new baseplates was 16 weeks.

Figure 5: ***New baseplates***

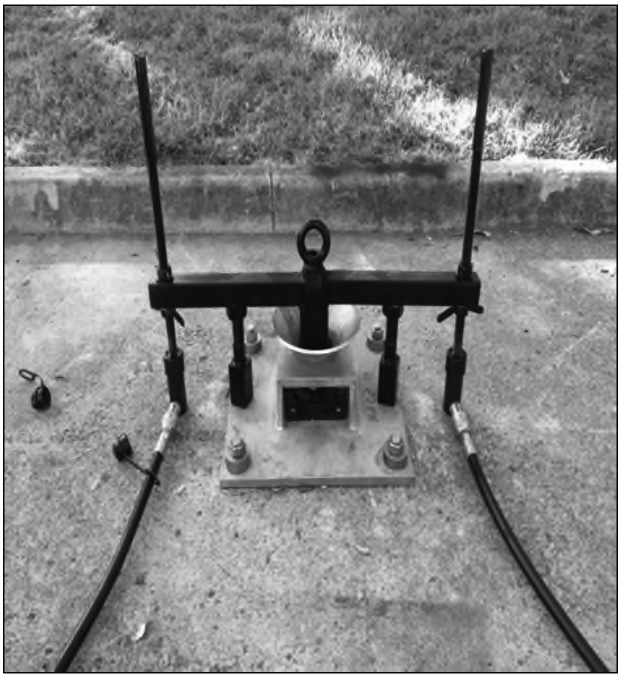
Upon arrival of the new baseplates, it became clear the proposed option of taking the bioreactor offline would not be possible. At this point I was looking at the new baseplates and thought "I could make a tool to lift them within the bioreactor". Having the new baseplates on site made it easy to come up with a solution as there was something to "touch and feel". Prior to the arrival of the new baseplates, only engineering drawings were available.

A meeting with the on-site fitters was held and we started designing the lifting tool using a new baseplate as a reference point. After the lifting tool was constructed, a baseplate was bolted and glued (using Sika Flex), to the concrete in the car park. We did this to operate and test the tool as close as possible to the real application. The testing scenario was successful and became useful when divers were engaged. The dive crew could see the tool in operation prior to the actual replacement task commencing, which was extremely beneficial as there is zero visibility at the bottom of the bioreactor.

The final result was a lifting device which was lowered to the base plate using the onsite jib crane. A diver secured the lifting tool to the worn base plate using a locking pin. The height of the lifting tool could be adjusted with wing nuts and a hydraulic ram was fitted to each end of the lifting tool connected to two hydraulic pumps. Each pump had a visual gauge so the operator could watch the hydraulic pressure and maintain even pressure on each ram (see Figure 6).



<u>Figure 6:</u> ***Lifting tool***



<u>Figure 7:</u> ***Lifting tool test***

The idea was to make the tool as easy as possible for the diver to operate. When the lifting tool was fitted, the diver moved away while jacking took place. The diver had full audio communication with the fitters above, which was essential for the safety and accuracy of the project. The hydraulic rams on the lifting tool had 25mm travel so a few ram height adjustments were required to clear the baseplate of the anchor bolts. The diver could feel if the baseplate was lifting off square and re adjust the wing nuts after each lift. When the plate was jacked off the anchor bolts, the lifting tool and base plate could be raised to the surface using the jib crane. This was possible as the lifting tool was still attached to the base plate with the locking pin (see Figure 8).

Figure 8: ***Removal of first baseplate attached to jib crane***

The day came to put the lifting tool to work. We had planned the project to take 5 days. The first base plate was removed before morning tea, and there was relief to see the base plate was as worn as expected and was in need of replacement.

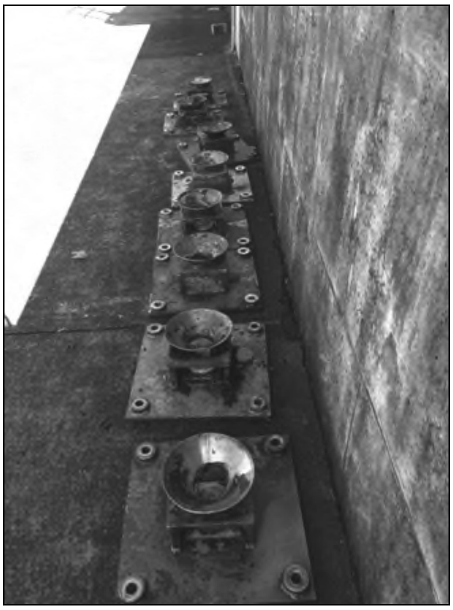


Figure 9: ***Worn baseplates removed from service***

All 14 baseplates were completed in 3 and a half days, with varying levels of wear depending on the load they were subject to.

4.0 CONCLUSION

Without the lifting tool, the baseplates could not have been removed without taking the bioreactor offline. All 14 baseplates have now been replaced and will be in service for the next 2 decades. The replacement project was estimated to cost in excess of $160,000 if the bioreactor was taken offline. By using the baseplate removal tool, the project was completed for less than $40,000.

BLOWER TECHNOLOGIES AND ENERGY EFFICIENCY OPPORTUNITIES

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BLOWER TECHNOLOGIES AND ENERGY EFFICIENCY OPPORTUNITIES

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ABSTRACT

The operating principles of today's Wastewater Treatment Plants (WWTP) are permanently under review against world best practice. Not only does the right treatment selection play a vital role, but the energy efficiency is also of utmost importance with energy consumption a major cost factor during the lifetime cycle of installed equipment. It is no longer good enough to purchase individual components. Plant managers, design engineers and consultants must work together to find world best practice system solutions for today's needs that flexibility cater for future requirements. With ever increasing energy costs, it is vital to understand the importance of the working principles of compressed air equipment, and its influence on the overall plant performance and lifecycle cost. Knowing that energy is the single highest operating cost in a WWTP, where blowers for aeration play a crucial role, knowledge of the different blower operating principles is important.

This paper outlines; the traditional approach to WWTP design and its associated efficiency problems, system solutions and an alternate design approach with key efficiency gains, guidance on how to calculate specific power for an individual unit and an entire system.

1.0 INTRODUCTION

The needs of commercial and municipal WWTP's vary substantially in size and projected period of planned operation cycle. It is not unusual that the designs for municipal plants are based on projected populations and demand 10 to 30 years in the future. As project funding is available now and may not be there later on, system engineers must build a system that will continue to serve the community's growing needs, in the most cost-effective, energy-efficient way possible. This is no simple task. Although the volume of air and pressure changes seasonally, each day, even hour to hour, the general practice is to design the plant's capacity for the worst case, maximum future load, resulting in oversizing the blowers. However, oversized blowers do not operate at their most efficient design point and spend as much as 90% of their operating time wasting costly energy.

2.0 DISCUSSION

2.1 Blower Operating Principle Efficiency

There are two main groups of compression principles; compression by dynamic and displacement design. An overview of the different operating designs available is shown in Figure 1. Standard for all applications with pressure requirements below 1000 $mbar_{(g)}$ is roots type blowers - a robust design where the compression process happens in the associated pipework. Displacement rotary blowers are of oil free design and so require clearance between blower housing and both rotating rotors. This results in slip (backflow) of air, resulting in reduced efficiency.

Figure 2 illustrates the differences between external and internal compression. With increased pressure influence, system engineers do their best to combat wasted energy by selecting energy efficient equipment.

This has led to an increased focus on energy and has helped spur innovations in blower technology. It is important to make the right decision on the blower package and its operating principle.

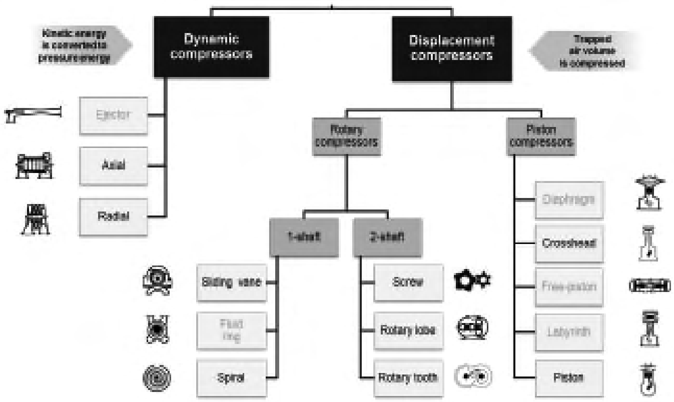


Figure 1: ***The various designs on compression principles for gaseous mediums***

External compression: Air is pushed out against the pressure losses in the process and therefore compressed in the piping between blower and application:

Internal compression: a certain pressure is always built up, independent of the process requirements:

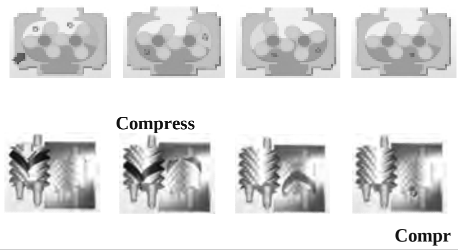
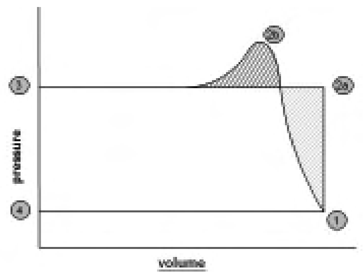


Figure 2: ***Compression principle three lube rotary blowers with pre-inlet channel design and screw blower***

Blower manufacturers are taking advantage of the increasing interest in energy savings and offer low pressure equipment in the form of Roots Type Three Lobe Blowers, as well as Screw Type Blowers. The question therefore is, 'what blower type for what application?' The trade off on work required to compress a gas, using internal or external compression principle is shown in Figure 3: an energy advantage of internal compression at precise adjustment of the screw geometry (green area), compared to additional consumption at over-compression (top area).



4-1: suction process
1-2b: polytropic compression screw profile
1-2a: isochoric compression blower
2-3: pushing out against pressure

Figure 3: ***Comparison on required work associated with blower using internal and external compression***

Energy savings are achieved as long as the area enclosed by 4-1-2b-3-4 is less than the area 4-1-2a-3-4. When choosing a blower with internal compression (helical rotors), the operating point must exactly comply with the by design (geometrically) determined optimum design pressure or else this will lead to over-compression and unnecessary energy consumption.

The internal compression of screw blowers operating in idle load, is always connected to higher power consumption. Pressure requirements of 500 $mbar_{(g)}$ and above should be considered as a base line where the use of screw blowers can be more efficient.

2.2 Plug & Play Solutions

The next step is “Wire-To-Air” efficiency. Here, the drive system, motor and starter, are carefully matched, ensuring minimum mechanical and electrical losses. A basic tool for the plant operation and system design engineer is simply total energy used to provide the specified flow and pressure and is expressed as a ratio of the power to the flow. While this metric is relatively new to the blower market, it is widely used for industrial compressors and compressed air systems and is often referred to as *specific power*.

Whether using the term plug & play, wire-to-air or specific power, it is important to differentiate between each individual piece of equipment’s efficiency and the overall *system* efficiency. The traditional system design approach for WWTP’s focuses on individual blowers instead of considering how each piece will work with one another. However, even if the most energy efficient blowers are selected, if they are not properly applied and controlled, they will not yield the anticipated energy savings. This is why system specific power is crucial in system design.

2.3 Specific Power Explained

In its most basic form, specific power is a product of input kilowatt to the machine divided by cubic metre per minute of air at standard conditions.

$$\text{Specific Power} = \frac{\text{Input kW}}{m^3/\text{min}}$$

While the equation is relatively simple, the process for calculating the value is not, no unified testing standard currently available to serve as a baseline for the calculations. There are a number of international standards manufacturers can use for determining equipment efficiency, depending on the compression principle of the equipment. Most commonly used is ISO1217-2008 annex C for compressors, operating on displacement principle. Furthermore, these international standards don’t use a common baseline, or unified protocol for manufacturers to publish their performance on datasheets. Because the testing procedures aren’t standardised, end-users aren’t able to make true ‘apples-to-apples’ comparisons when considering different equipment to purchase.

As there is currently no testing standard and no published datasheets for blowers, the burden of making uniform calculations is on the user. To calculate specific power, it is possible to measure the input kW at the package control panel and install a flow meter at the outlet of the package to determine flow. However, when specifying equipment, this is not possible as the equipment is not on hand.

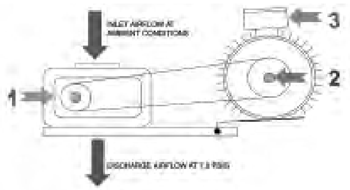


Figure 4: ***Blower consumption (1), drive losses (2), and motor efficiency (3), all impact specific power***

This necessitates understanding how to calculate specific power for the entire package and how each component will affect the overall efficiency.

Looking at the specific power of only a blower block from one manufacturer versus that of an entire package from another will not give a true efficiency comparison. To calculate specific power, first the flow rate needs to be determined. For positive displacement blowers, this is a function of the blower's displacement per revolution, blower slip, operating RPM, ambient conditions, and operating pressure.

$$\dot{V}_1 = V_0 \left(n - n_{slip} \cdot \sqrt{0.0371 \cdot \frac{T_1 \cdot \Delta p}{p_1 \cdot 1K}} \right)$$

V_1 =	*Suction Volume*	n_{slip} =	*Slip Speed (basis 100mbar)*	*p1 = Inlet Pressure*
V_0 =	*Displacement/Revolution*	*T1* =	*Inlet Temperature in Kelvin [K]*	
n =	*Block Speed*	Δp =	*Pressure Differential*	

Next, we need the required blower power (1) which is a function of mechanical design and pressure differential. As ambient conditions are the same as normal conditions in this example, Nm^3/min and m^3/min are equal. Now we have blower power (1), but this is not what the user is paying for. The user is paying for electrical input at the motor (3).

To calculate motor input power (2), we need to determine the losses associated with the drive. For most v-belt slide base designs where the motor can be moved to adjust centre distance and apply tension, we can expect a 5% loss. For more advanced tension systems, these losses can be reduced to 2-3%. Finally, we need the rated motor efficiency as given on the motor nameplate. Therefore, input kilowatt (3) is given by;

$$\frac{\text{Blower Horsepower} \times (1 + \text{Drive Efficiency}) \times 0.746}{\text{Motor Efficiency}}$$

Once input power is calculated the specific power can be obtained.

$$\text{Specific Power} = \frac{\text{Input kW}}{m^3/min}$$

The lower the specific power value, the more efficient the blower is. Here we only evaluated the blower, belt drive, and motor, and we assumed ambient conditions to be the same as standard conditions, which helped simplify the flow values and calculations. In reality, most blower systems include accessory components such as silencers, filters, and valves, which all present flow restrictions. Flow restrictions result in a greater pressure differential across the blower and result in more power consumption. In addition, other package designs utilise cooling fans (shaft or separate), pumps for cooling, or some other electrical or mechanical device, which add to the power requirements of the machine. We also need to consider the different internal pressure between rotary lobe and screw blowers. For the best accuracy, input kilowatt should be measured at the input of the machine's control panel. This takes into account all losses associated with the package as well as other relevant components. In addition to system losses and power consumers inside the package, power consumers in the control panel also need to be considered. The sum of each of these gives the total package input kilowatt consumption.

So far we have evaluated the elements of a blower package, power transmission, and accessory power consumers to represent the performance of the physical package. For a fixed speed machine, the specific performance of the machine is mostly constant (excluding the effects of ambient conditions). However, the vast majority of modern wastewater systems utilise variable frequency drives and the demand is split between the units enabling handling flow and pressure requirement at constantly changing needs.

2.4 Variable Frequency Drive

Variable frequency drives (VFDs) allow equipment to operate at different speeds by adjusting the voltage and frequency delivered to the motor. This gives the machine versatility by varying blower performance to match system demand; however, this comes at a price. Most variable frequency drives have an efficiency rating just like motors. The 97% VFD and 95% motor efficiency do not apply when the unit is running at ¼ or ½ speed. At these reduced speeds, the efficiency is decreased; therefore, VFD usage should be limited to applications where the demand actually fluctuates.

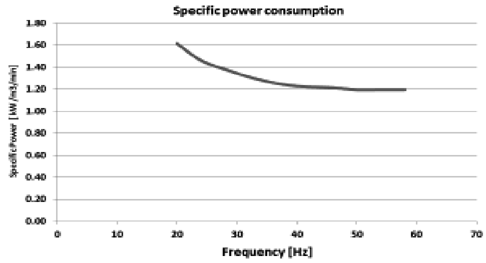


Figure 5: ***Running at reduced capacity greatly impacts the energy efficiency of a variable frequency drive unit***

VFD's are beneficial in handling fluctuations in demand, especially when compared to blowing off excess air to atmosphere. What should be avoided, however, is using a VFD on an oversized machine.

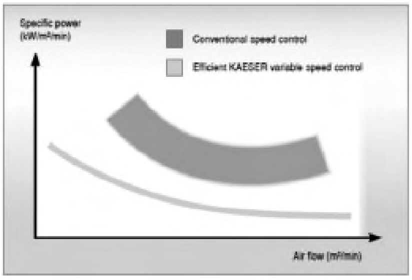


Figure 6: ***Specific power consumption conventional speed control and blower manufacturer speed control***

2.5 Focus on the System

For a multiple blower system, the focus must be on system efficiency. It is not enough to simply use the most efficient blowers. Well-designed blowers are a great start but to operate as an efficient system, they must be applied correctly and controlled properly.

Now that we have examined the efficiency for individual units, we can apply those concepts to understanding the overall efficiency of the entire system and how sizing and selection affects the efficiency of the system. This means, system design engineers should actively investigate the possibility of using rotary lobe- and screw- blowers in one system. Conventional aeration system designs include two large blowers. This needs to be critically reviewed, knowing that WWTP's are designed for the future, and that for a long time blowers are underutilised and operating most of the time in part load. Also the impact of large variable speed blowers with an identically sized back up unit, are the main reason for the high energy consumption. In-house basic control systems are set to share the demand and the units cost more to purchase, their drive losses must be factored into the unit's efficiency.

Flexibility and energy efficiency can be achieved by an alternate method of system design, so called system splitting. With system splitting, the maximum load is split among several cycling online/offline fixed speed machines to cover the large portion of the demand and a variable frequency drive (VFD) machine to cover the trim load. This method of system design allows much more efficient control without sacrificing the ability to meet the occasional periods of higher demand. Simulation programs can assist working out the best combination on rotary lobe blowers, screw blower and which one should be driven by variable speed drive.

2.6 Adaptive Control

For system splitting, only one or two machines are VFD units. If there are two VFDs in the system, only one runs at a time, with the second acting as back-up. The remaining blowers are fixed speed units. By limiting the number of VFD units in the system, initial investment costs are considerably lowered. The final component of system splitting is controls. Adding an adaptive master controller makes it possible to find the best combination of units to meet the current demand. Since the fixed speed units run on auto-dual control, the units can run idle for a defined period of time before shutting down. This gives the adaptive master controller enough time to observe the system's response and signal the units to reload if needed. The VFD is sized no larger than required, reducing the initial investment cost while covering the supply gaps that occur when the fixed speed machines are offline.

Adaptability: A master controller that learns the system and adapts to fluctuating demand can better respond and choose the most efficient unit combination to meet the demand and improve pressure stability.
Integration: Chances are you have a plant SCADA system for monitoring. Look for a master controller with communications capabilities that will easily integrate into what you already have.
Back-up: The right controller can help reduce maintenance costs, with some able to rotate like-sized machines to equalise run times and spread out maintenance intervals. Some will also let you specify the units to run as back up only, ideal where you want older/less efficient units to only operate if a unit fails.

Figure 7: ***Adaptive master controller; key criteria to consider when selecting a controller***

3.0 CONCLUSION

Wastewater treatment is a critical utility and the system must be designed to reliably meet its highest expected load. When it comes to the blower system, bigger is not always better. The best air system design is a holistic one that takes into account the range of demand, future growth, the entire system's specific power, and optimised energy efficiency. System splitting, using rotary lobe and screw blowers, and using an adaptive control scheme can provide reliable supply without unnecessarily burdening the community with higher energy costs. Understanding system dynamics can save initial costs as well as maintenance and power costs for many years to come.

4.0 ACKNOWLEDGEMENTS

Co-authors; Albin Hess & Stephen Home, KAESER Compressors

OPERATIONAL CHALLENGES FOR THE DISINFECTION OF REPLACEMENT FILTRATION MEDIA

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Simon Woodford, *Operator Supply (Sunshine Coast),*

Seqwater

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OPERATIONAL CHALLENGES FOR THE DISINFECTION OF REPLACEMENT FILTRATION MEDIA

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Simon Woodford, *Operator Supply (Sunshine Coast),* Seqwater

ABSTRACT

Seqwater recently implemented a number of disinfection procedures for maintenance and project works. This project critically examined the disinfection procedure for filtration media which was based upon the AWWA standards B100 and C653. Sites that applied a single batch of high-chlorine solution consistent with the AWWA Standard were unsuccessful in keeping chlorine residuals above the required 15 mg/L, even when the process was repeated with higher starting concentrations. The current study identified improvements to manage this issue and ensure timely reinstatement of the filter.

These challenges were successfully managed at two of Seqwater's sites through the continuous flow of disinfectant through the filters using on-site dosing equipment. This included the (1) backwash system at North Pine WTP and (2) filter to waste configuration at Noosa WTP. In both instances, chlorine demand was eliminated within a few hours before retention for a further 12 hours. Upon draining, chlorine residuals were measured and it was found that water above the media and in the under-drains retained the chlorine concentration with little decay (residuals >100 mg/L). However, fractions within the top section of the media bed in contact with the finer particles and anthracite were much lower (e.g. 30-100 mg/L) but remained above 15 mg/L.

1.0 INTRODUCTION

Seqwater is the bulk water authority in South East Queensland, responsible for 37 treatment plants and more than 600 km of pipelines in an extensive bulk water supply grid that stretches across the region. The organisation formed through the amalgamation of three bulk water supply providers during the Queensland Water Reforms of 2008-2012. Through these mergers, Seqwater gained extensive knowledge and procedures for the management of disinfection following maintenance and project works on treated water assets such as reservoirs, mains, pumps and filters.

Implementation of the procedures involved a number of challenges that would be faced by many water service providers. This includes operational sites that can only be offline for short durations to ensure continuity of supply, and difficulties achieving the targeted disinfection C.t (i.e. chlorine concentration × contact time) due to the effect of excessive chlorine demand (e.g. filter media replacements). The current project focussed on the procedure for the effective disinfection of filters and filtration media.

There are a number of internationally reputable standards for filter media replacement and the disinfection practices to prevent microbial contamination of drinking water upon reinstatement. The standards reviewed in this project included the following American Water Works Association (AWWA):

- AWWA B100-09 Granular Filter Material
- AWWA C653-13 Disinfection of Water Treatment Plants.

In summary, these standards require the retention of a free chlorine residual of 25 mg/L, held in the filter and its media at not less than 15 mg/L for 12 hours.

This is important to ensure safe drinking water as pathogens may have been introduced during the filter refurbishment and media replacement. The Australian Drinking Water Guidelines (2011) identifies a number of pathogenic protozoa, such as *Cryptosporidium* and *Giardia* that have a resistance to the chlorine levels in treated water and therefore the higher chlorine concentrations and contact times in these standards needs to be achieved to ensure adequate inactivation of these pathogens.

The use of retention methods for chlorine disinfection where the resultant water is able to be disposed or removed from the drinking water supply system can allow the use of high concentrations of chlorine sufficient to inactivate resistant protozoa. However, it had been observed previously in the filter refurbishment at another large operation that this was difficult due to the excessive chlorine demand that exists in either completely new or topped up media. Despite the use of increasingly high starting residuals, they were not able to be kept above the 15 mg/L required in the above AWWA standards at the end of the 12 hour retention period.

The disposal of large quantities of chlorinated water is also a challenge. The return of chlorinated water to the head of the plant (e.g. via supernatant return) is not possible as it increases the risk of creating disinfection by-products such as trihalomethanes in the water supply. If disposed to a receiving environment, chlorinated water needs to be neutralised of its chlorine residual in order to ensure environmental obligations and site licence conditions are met. This is important if the volume of water used to achieve disinfection is sufficient to run-off or overflow from site.

The project considered these issues and critically examined the Seqwater procedure for the disinfection of filters and filtration media. A number of changes were then trialled with the support of process engineers at two relatively large water treatment plants at North Pine (Lake Samsonvale) and Noosa (Lake MacDonald).

2.0 DISCUSSION

The issue of excessive chlorine demand when applying the disinfection procedure at other sites demonstrated that only regular replacement of the disinfectant solution or a continuous flow of the disinfectant would be sufficient to eliminate chlorine demand before any static retention period when undertaking filter media disinfection. The project at North Pine involved the disinfection of individual filters using the continuous flow of disinfectant water through the backwashing system in order to deplete chlorine demand followed by a retention method of 12 hours. Similarly, the project at Noosa applied a continuous flow approach, but the chlorine was injected into the settled water supplying a bank of three filters and subsequently filtered to waste.

During the process, chlorine residual samples were collected to determine when chlorine demand had been met and when the retention period could begin. At the end of the retention period at North Pine Water Treatment Plant, samples were collected during drain-down to examine the various concentrations of chlorine that remained in the different parts of the media bed (i.e. anthracite, sand, gravel and under-drains).

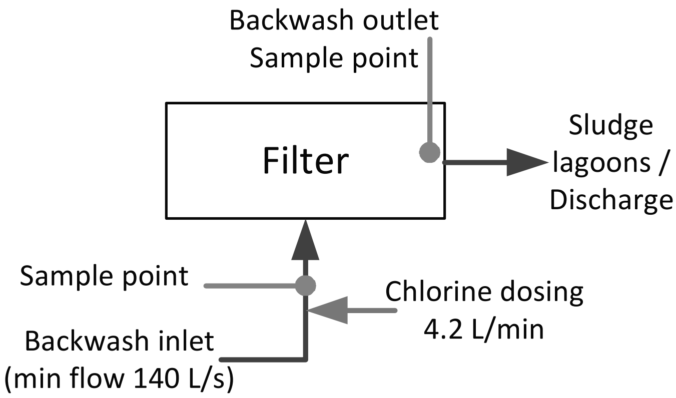
2.1 Filter Disinfection at North Pine Water Treatment Plant

Recently, North Pine has gone through a refurbishment of its five filters including the replacement of the media.

Obstacles required to be overcome during reinstatement included the dosing of sodium hypochlorite (the disinfectant) into the backwash main (Figure 1), the ability to have the other four filters online/backwashed at the time of disinfection and the ability to control the flow into the filter using a high level reservoir and a control valve.

<u>Figure 1:</u> ***Dosing disinfectant into the backwash main at North Pine***

The process used is shown in Figure 2. The backwash valve was set to a position reducing flow to 140 L/s. This was the lowest possible flow into the filter because of flow meter limitations. A temporary line was run from the chlorine dosing system to an existing injection point on the backwash main and operated up to a maximum flow rate of 4.2 L/min.



<u>Figure 2:</u> ***Disinfection by backwashing at North Pine Water Treatment Plant***

Originally, the first filter was disinfected at lower dose rates, but it was observed the concentration in the backwash discharge could rapidly drop away to as low 16 mg/L then rise back over 30 mg/L in space of 30 minutes. Starting at 4.2 L/min allowed the operator to not be concerned by this issue of the new filter media demand stripping concentration away from the desired target. This would otherwise have been time consuming and had the potential to require the procedure to be repeated. The dose rate was then reduced to 3.1 L/min as the concentration out of the backwash discharge started to rise higher above the target concentration. The hypo pump dose was set between 3.1 and 3.4 L/min for the remainder of continuous flow disinfection. Towards the end of this run (last hour) the dose was put to 4.4 L/min to ensure the high concentration was throughout each layer of sand, gravel and anthracite over the 12 hour static disinfection. This is shown in Figure 3. The dose was determined using the results of grab samples out of the backwash discharge after the highly disinfected water had travelled through the filter.

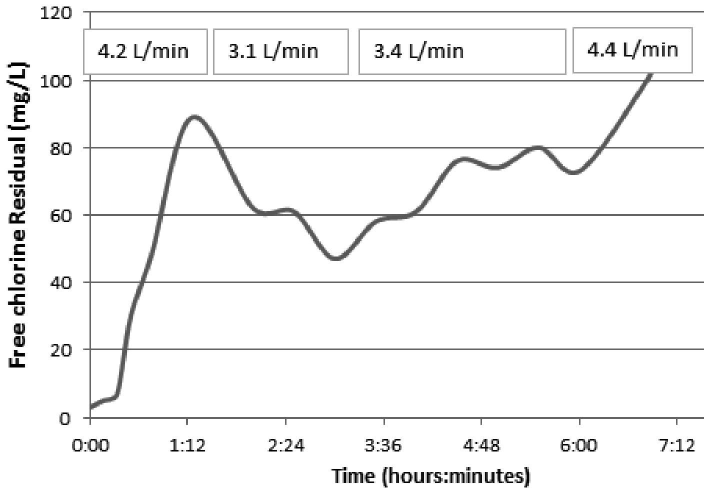


Figure 3: ***Chlorine demand observed in filter disinfection at North Pine***

Disposal of the high concentration backwash water was managed through the sludge disposal system at North Pine. The water was directed through the sludge clarifiers, a sludge lagoon and the final polishing lagoon which discharges to the river. Continuous monitoring of this discharge confirmed that this was within environmental licence requirements of 0.02 mg/L. Approximately 3 ML of water was used during this process.

2.2 Filter Disinfection at Noosa Water Treatment Plant

The Noosa project's first priority was to determine how to achieve the chlorine dosing into the filters. Options included manual dosing directly into the top of filter (e.g. using an IBC) or using the existing sodium hypochlorite dosing system into the settled water channel. It was determined the existing dosing system was the easiest option as the operator could manually adjust the filter outlet valve to obtain a flow rate of 40 L/s and the dose rate calculation for chlorine could achieve more than 80 mg/L to the top of the filters.

The project's first attempt took the approach to use the minimum chlorine dose rates required into all three filters. This method was chosen because there was an unknown factor of our demand in the filters and the project team did not want to waste chemicals or exceed the environmental disposal considerations. The project should have started with an extremely high dose rate to achieve the desired 30 mg/L at the filter outlet valve and this would have increased the desired chlorine residual in the filtered water despite the significant demand present.

The project's second attempt was conducted as shown in Figure 4. The approach was to dose a very high chlorine rate into each filter at the time and target 60 mg/L at the outlet valve. Initially the chlorine residual in the outlet valve would drop away. However, once the operator had dosed all three filters to obtain 60 mg/L at each outlet valve, then all three filters were operated at the same time. The combined filtered water chlorine levels were rechecked to determine an outlet residual of above 30 mg/L, due to the decay during the three hours it took to dose the filters one at a time.

Once stable, the dosing process was ceased and the plant was locked out overnight. It was calculated that 12 hours total contact time should have been acceptable to maintain above 15 mg/L in all three filters and this was confirmed when sampling was conducted the following morning and 18 mg/L was achieved.

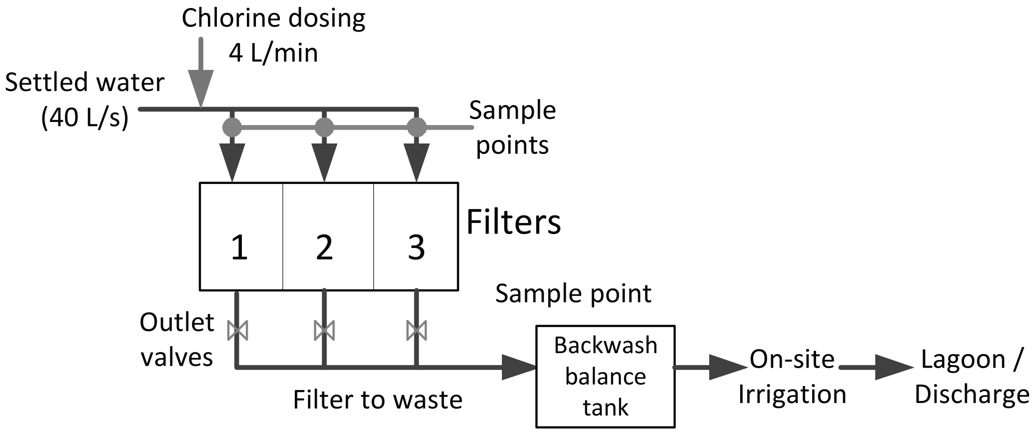


Figure 4: ***Disinfection by filtering to waste at Noosa Water Treatment Plant***

Another factor the project team had to consider was the supernatant was to be released to the environment through the backwash recovery system. The project team decided to blend the filter to waste water from this project with the plant sludge bleeds and Biological Activated Carbon (BAC) filter backwashes that were on line at the time as the parallel trains in the plant were still producing treated water. The water was then irrigated using a portable pump to a large grassy area before our designated supernatant lagoon which releases to the environment. This was found to be effective in reducing the chlorine levels to within the site's environmental discharge licence requirements before reaching the lagoon.

2.3 Final Chlorine Residuals in Different Layers of the Media Bed

The massive amount of surface area on the sand particles in the media bed created a significant chlorine demand that needed to be eliminated in order to sustain sufficient chlorine concentrations for the necessary contact time. Backwashing new filtration media several times to remove the fines did not sufficiently clean these surfaces. Upon draining down at the end of the 12 hour retention period for one of the North Pine filters, chlorine residuals were measured at intervals representative of different layers in the filter bed. The results are shown in Table 1.

Table 1: ***Filter media layers and chlorine residuals***

Approximate fraction	Free chlorine residual (mg/L)
Top of filter (above media)	97-108
Top 0.3 m (anthracite and sand)	36-36
Centre 0.3 m (sand)	108
Bottom 0.3 m (sand-gravel)	167
Bottom of filter (under drains)	171

It was found that surface water below the media retained the chlorine concentration with little decay (>100 mg/L). However, fractions within the media bed in contact with the finer particles and anthracite (obtained during drain down) were much lower (for e.g. 30-100 mg/L) but remained above the 15 mg/L specified in the AWWA standard. This clearly showed the extent of chlorine demand in the fine sand and even more so, the anthracite layers compared to the rest of the filter infrastructure and media.

2.4 Recommendations for Future Filter Refurbishment Projects

The project made the following recommendations:

- The use of existing dosing equipment to inject chlorine into the backwash or settled water to provide control and avoid any unnecessary manual handling or chemical exposure for operators.
- The continuous flow of disinfectant is critical in removing most of the chlorine demand before the retention period required by the standard procedure.
- The high use of chlorine and water to waste is a worthwhile investment to ensure the filter is brought back online quickly and the procedure does not have to be repeated, thereby resulting in multiple days of down-time.
- If required, the refurbishment and disinfection should be conducted so that the remaining filters or parallel treatment trains can continue to operate.
- The depletion of chlorine levels in discharged water in order to meet the environmental licence obligations is best achieved by mixing in backwash recovery or sludge handling system; otherwise the use of neutralising agents such as ascorbic acid tablets will need to be considered.

3.0 CONCLUSION

The project found that (1) the continuous flow of the disinfectant through the media bed was essential in successfully achieving the minimum chlorine residual requirements in all of the media layers and (2) the approach was effective in returning the filter to service in the minimum time necessary without the risk of having to repeat the procedure.

Additionally, options for a number of operational challenges in disinfecting filters were identified for future filter refurbishment projects. These included using maximum dose rates through existing dosing equipment with minimum plant flows, suitable dosing points on either backwash mains or settled water channels, frequent operator testing and adjustments, and neutralising chlorine residuals in when disposing water to the environment.

4.0 ACKNOWLEDGEMENTS

The author is appreciative of Michael Burns and Mark Wilson for the process engineering support that helped with the project. The construction managers, Jason Keune (North Pine) and Dale Anderson (Noosa) are kindly acknowledged for their cooperation with the operations team throughout the filter refurbishment projects.

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INFLOW AND INFILTRATION

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42nd Annual WIOA
Queensland Water Industry Operations Conference and Exhibition
Logan Metro Sports Centre,
Crestmead
7 & 8 June, 2017

INFLOW AND INFILTRATION

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ABSTRACT

Rain events that cause Inflow & Infiltration (I&I) have enormous impacts on the operation of wastewater networks and treatment plants. To meet our regulatory obligations and licensing requirements, a detailed analysis and modelling project of the Gold Coast Water & Waste (GCWW) operational network performance was implemented in a more focused direction. Whilst history tells us that the Gold Coast catchments will endure 2 to 3 high impact events per year, it is hard to put a monetary value on the true budget cost of these events.

This project will not only identify GCWW's analysis of pump station performance, emergency response, network improvements but also the requirement for not putting maintenance personnel and the public at risk during storm events.

1.0 INTRODUCTION

With population growth of approx. 10,000 people/yr, new housing expansion has been an extensive part of Gold Coast's development. The Gold Coast's geography covers 1338 km^2 of flat landscape, with a vast majority being swamp reclaimed land, sand developments and canal estates. The Gold Coast has a mean rainfall of 1337 mm/yr. With an aging network of pipes constructed in Asbestos Cement (AC), Vitrified Clay (VC) and Polyvinyl Chloride (PVC) and high water table areas, inflow & infiltration (I&I) is inevitable. Construction of house drainage and pipeline design techniques has remained greatly unchanged for decades. GCWW also has 4 vacuum sewer networks which have highlighted a completely different set of I&I issues to our conventional networks. GCWW operates 4 wastewater treatment plants, 529 wastewater pumping stations (WPS) and 3,210 km of network.

I&I has a major impact on services provided by GCWW. In the last 2 years GCWW has had one 1 in 50 and two 1 in 20 year rain events occurring in January, February 2015 and March 2017. The impact of these events had an extreme effect on the operation of the network. Spills occurred over a 48 hour period from direct inflow and then subsequently the infiltration. Over the years, GCWW has managed rain events by pumping excess flows to treatment plants, controlled using tankers, constructed designated overflows in impacted areas or the system relieves pressure at undesignated locations i.e. maintenance holes or Overflow Relief Gullies (ORG's).

2.0 DISCUSSION

2.1 Impact of Inflow and Infiltration

The impact of direct inflow on wastewater networks on the Gold Coast is much more dynamic, as it has immediate impact on pipelines and pump stations. It is not just rain events that can catch us out but the high tides, canals estates and creeks/rivers entering our network. Illegal stormwater connections are a hidden form of inflow.

Infiltration is a hidden quantity. It can be there and go unnoticed for years, unless the right technologies are applied, implemented and analysed. Infiltration can be from several sources. Creeks, canals, high water tables and tides enter through cracks, broken pipes, damaged house services and ORG's. As stated, I&I impact's every part of the network.

But how do you single out just one as the worst! Effects of I&I is widespread, from having spills in customer properties or other areas, and treatment plants overflowing to the environment. I&I can have significant health and environmental, particularly in public access areas such as water ways and beaches that the Gold Coast is renowned for.

How do you budget for unforeseen events and their associated contingency plans? Tens of thousands, if not millions of dollars are spent in transportation of the excess flows, maintaining and improving the network, and the clean ups.

Customers have an expectation that GCWW will deliver them a safe, secure and reliable service.

2.2 Identifying Inflow & Infiltration

Obtaining a first indication of I&I within our system is by recording the inflow into our 4 wastewater treatment plants. The operations for each plant before and during a 1 in 50 year rain event on 24th January 2015, is detailed in Table 1.

Table 1: ***Flow data for treatment plants***

Treatment Plant	Dry weather flows	Rainfall mm	Wet weather flow
Pimpama	6.5 ML/day	113mm	29.3 ML/day
Coombabah	75 ML/day	187.4mm	251.50 ML/day
Merrimac	32 ML/day	271mm	131.3 ML/day
Elanora	18 ML/day	291mm	84 ML/day

Total wet weather treatment plant flows – 496.6ML
Water consumption 24th January 2015 - 165ML

GCWW has a total of 529 WPSs all of which are monitored by SCADA. Historic data from the wastewater network can be archived for 4 years on the data network and is then archived on our mainstream system. SCADA is our front line of information to allow us to identify I&I impacted areas. This information can be quickly accessed through our SCADA trends to identify the level of impact on a station and also pump performance.

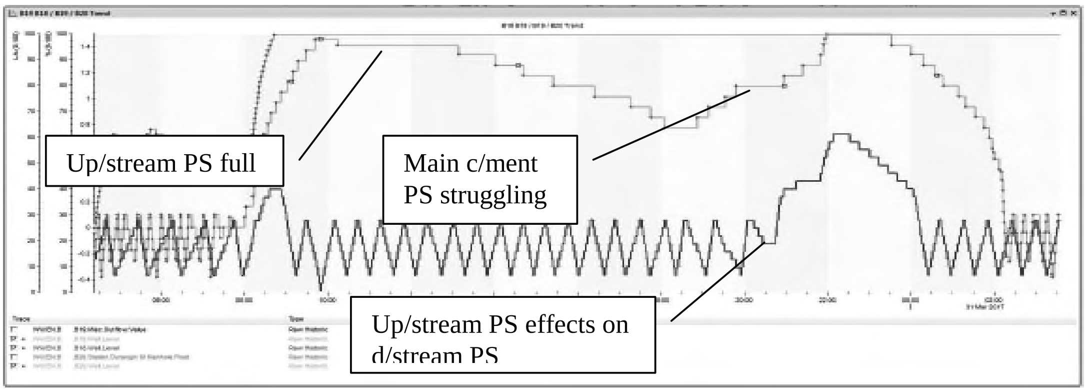


Figure 1: ***Graph showing inflow impacting 3 wastewater pumping stations***

Further dissection is able to view multiple catchments using historic trending data OSI PI software purchased by GCWW. This allows extracting large amounts of data to analyse and model large catchments for possible I&I network impact.

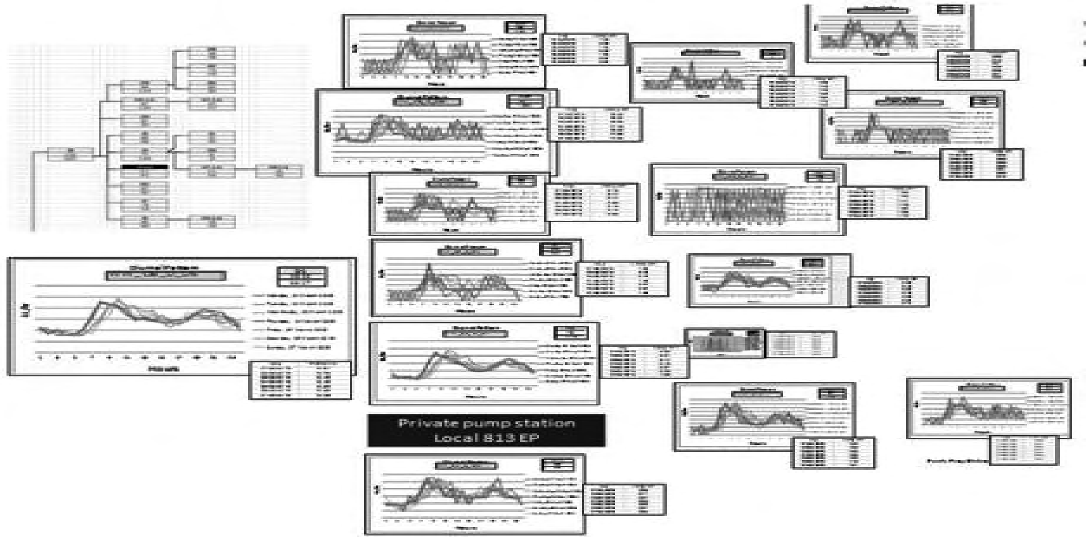


Figure 2: ***Analysing with OSI PI large catchment areas within the network***

A prime indicator of infiltration in and around low lying catchments is to view WPS performance between the hours of 00:00 and 04:00. This identifies those catchments with excessive pump runs/hours and infiltration during this period. Notably we can analyse industrial catchments which are non-operational and residential areas during this period.

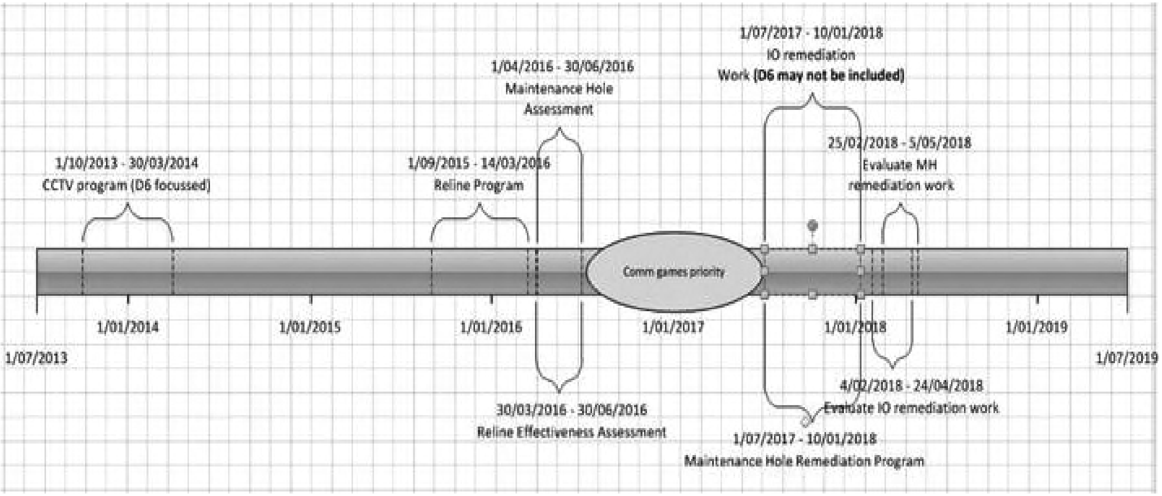
Pump stations located on tidal verges, which can be up 2-3 km's inland can also indicate infiltration from tidal water. During our high season, January through to March, where tides heights have reached over 2 m, significant I&I has been recorded. In some cases, Electrical Conductivity (EC) of up to 38,000 µs/cm have been observed and we have also detected direct inflow from spill points. Testing for EC occurs in these areas, particularly in the high tidal season, using WPS online sensors or the old fashioned way of walking the pipelines until a section of pipeline is targeted.

What is also a very useful tool is the GIS spatial data of the wastewater network. From this GCWW has been able to identify and target the most susceptible catchments, including age of pipe, depth, water table vulnerability and tidal influences.

Figure 3: ***Using GIS network to identify pipe type, age and depth in high water table susceptible areas***

2.3 Project Direction

As an outcome of the intensive analysis that has driven the project scope. Fundamental actions have been considered and implemented.



<u>Figure 4:</u> ***Project direction scope for D6 catchment***

A CCTV program has been rolled out to investigate high impact catchments. A total of 155km of the network has been CCTV'd in 2015/2016, with approximately 12.5% found to have a defect. In 2016/2017 using a strategic approach to prioritising high risk areas of I&I, 200km has been CCTV'd with approximately 30% found to have issues requiring repair. This forms part of our proactive approach pipelines repairs. Relining is one method of repair going from manhole to manhole or alternatively it may require complete pipe replacement.

A manhole assessment project has been instigated to physically assess all manholes throughout the GCWW catchments. A total of 65,398 manholes are located within the Gold Coast area with 689 manholes assessed in the first two (2) years. Information collected to assess manholes includes identifying manhole condition, water ingress (walls and donuts), asset location and asset detail. This information is then used to prioritise our manhole repair work programs. Network flow in manholes is also critical in augmentation of the network and it is surprising to note how many network bottlenecks are caused due to manhole channel design and construction issues.

Flow monitoring equipment offering alternative methods of flow analysis and retrieval has been instrumental in ascertaining and verifying modelling data and I&I investigations. In the last 2.5 years, equipment has been sourced that allows all data to be uploaded via the internet and accessed by modellers and engineers, without the requirement of site visits to download data. This has allowed better utilisation of the equipment, quicker assessment of data to better implement whole of catchment real-time analysis and the development of improved solutions. Progress is now being made to implement all the data collected by SCADA to reside in OSI PI templates in the future, to allow full data manipulation.

This equipment has also been used to analyse the Metricon Stadium performance during an AFL game, in order to measure capacity for the Commonwealth games opening and closing ceremonies in 2018.

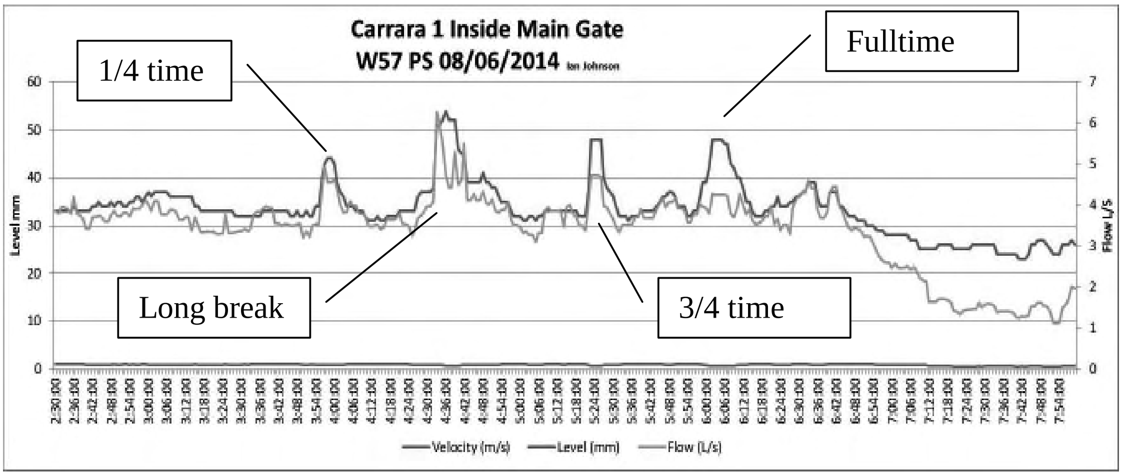


Figure 5: ***Commonwealth games flow trial from an AFL game at Metricon Stadium***

With online flow monitoring, we were able to identify silt build up in the pipe being monitored. This then instigated cleaning of the pipeline, thus increasing the efficiency and performance of the system as shown in Figure 6. Online monitoring also reveals data showing I&I, tidal flows, high night time diurnal flow patterns and under capacity mains.

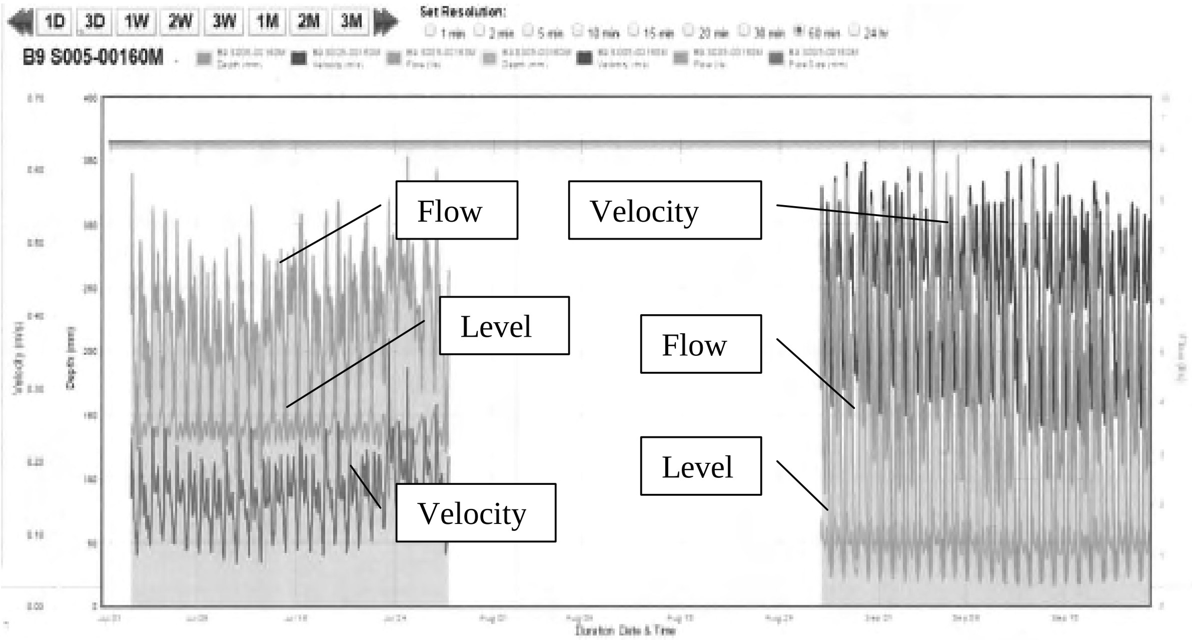


Figure 6: ***Data indicating before & after mains scouring***

This equipment has also allowed us to gain linear flow comparison data of how the liquids transgress along pipelines. This helps with being able to determine the flow speed in troublesome pipelines and identifying choke points within the network.

2.4 Previous Detection of Inflow and Infiltration

Smoke testing is highly labour intensive for little result in our climate, with high water tables, tidal influences and soil types impeding its effectiveness.

With the utilisation of rainwater tanks and also inline water traps on internal pipework, dye testing for illegal stormwater connections has also become difficult to perform.

Walking the systems whilst a good practice, is not a safe practice during rain events.

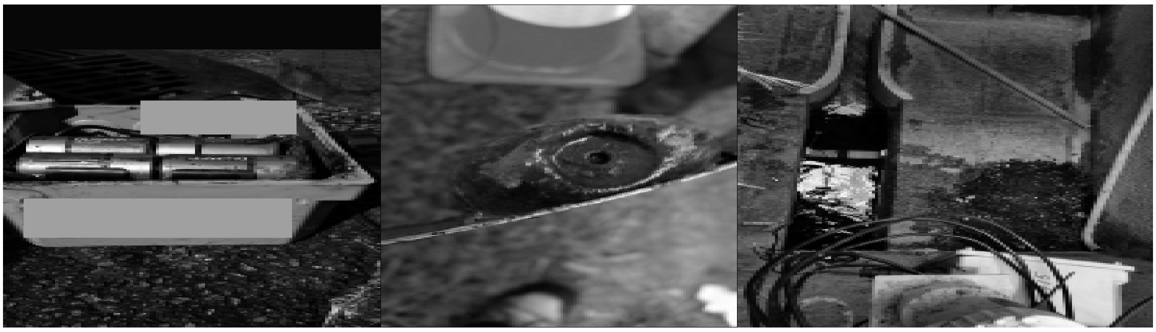


Figure 7: ***Flow monitoring is not without its failures***

2.5 Resources

Obtaining personnel with the skill to install and understand the fundamentals of I&I has proven to be an occupational hurdle. Investment is a must to educate and train operational personnel so future analysis of I&I can be continued and/or implemented. Having no experienced personnel will hinder any future projects. We all need to plan for succession.

3.0 CONCLUSION

The key points from this project are to:

- Continue collecting, analysing and ongoing monitoring of data from available resources including SCADA and field monitoring verification equipment.
- Improve data accuracy and retrieval. This will significantly enhance the accuracy of data modelling and network performance for the future.
- Prioritise catchment management identification of asset reviews to allow budgeting and resources to be centralised to better manage identifying operational improvements within the operating network, to systematically control the magnitude of I&I impact events.
- Implement systems that remove human manipulation for operational requirements of the network. Review what works and what doesn’t!
- Refrain from having segregation of knowledge within operational departments. Investigate new technology to improve network systems. Open your works for others’ education. Seek opportunities to gain knowledge from different authorities.
- Utilise WIOA as a portal?

I’m a realist, who has the billions of $$$$ to eliminate I&I? Importantly, we can learn to better manage it.

4.0 ACKNOWLEDGEMENTS

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Dominique Kierens – Senior Asset Investment Officer Service Sustainability

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Winner of the Best Operator Paper at the
79th Annual WIOA Victorian Water Industry Operations Conference,
Bendigo, 2016

INCIDENT AT YERING GORGE PUMPING STATION. HOW WHAT YOU DON'T KNOW CAN HURT YOU

Paper Presented by:

John deBoer

Author:

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Melbourne Water

42nd Annual WIOA
Queensland Water Industry Operations Conference and Exhibition
Logan Metro Sports Centre,
Crestmead
7 & 8 June, 2017

INCIDENT AT YERING GORGE PUMPING STATION. HOW WHAT YOU DON'T KNOW CAN HURT YOU

John deBoer, *Operations Specialist, Water Supply Operations,* Melbourne Water

ABSTRACT

On the 12th of May, 2015, a sequence events began that, 10 days later, would culminate in a major hydraulic incident leaving the Yering Gorge pumping station critically damaged and exposing the operator on site to significant risk. This event would result in the station being out of operation for two months before partial pumping could be reinstated and a total of eight months before the station would again operate at its full capacity. The ultimate cost of this incident included approximately 10 Gigalitres of lost water harvest and a significant restoration operation costing over $1.2 million. This paper describes the events leading up to the incident, the investigation process and the key learnings identified.

1.0 INTRODUCTION

The Sugarloaf Reservoir is a 96 Gigalitre storage located 35 Km North East of Melbourne in Christmas Hills and supplies potable water to the Northern suburbs via the Winneke Water Treatment Plant. The Yering Gorge pumping station delivers water from the Maroondah Aqueduct and the Yarra River into the Sugarloaf Reservoir and is the primary source of inflow to Sugarloaf.

The total pump station capacity is approximately 1100 ML/day and comprises of 4 main pumps each with a capacity of 250ML/day and two additional supplementary pumps each with a capacity of 40 ML/day. The station is operated to harvest approximately 200 ML/day from the aqueduct together with the maximum available river flow excluding the minimum required environmental passing flow of 350 ML/day.

2.0 DISCUSSION

2.1 Pump Station Configuration

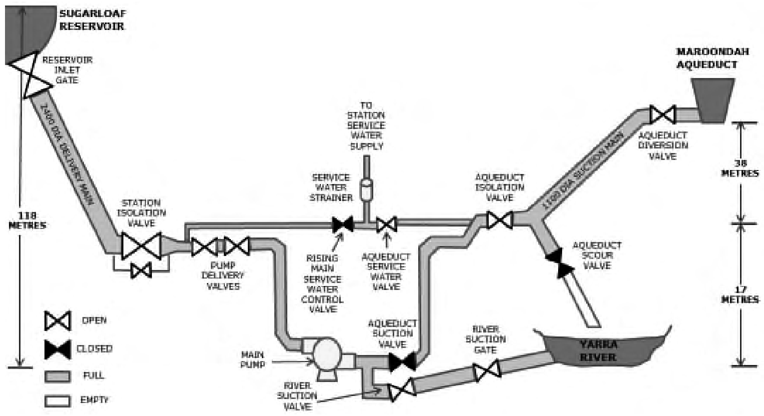
The Yering Gorge pump station can deliver water from either the Yarra River or the Maroondah aqueduct which is supplied from the Maroondah Reservoir in Healesville. The pump well is situated approximately three meters below the river bed and 55 meters below the Maroondah Aqueduct, with the station delivering water into Sugarloaf Reservoir which is approximately 120 meters above the pumping station. The aqueduct suction main is an 1100mm diameter pipeline which drops almost vertically from the aqueduct into the station below (see Figure 1). There is a manually operated diversion valve at the aqueduct, an actuated isolation valve on ground level 17 meters above the pump well, and an 1100mm actuated suction valve at each of the two aqueduct duty pumps.

The station draws water from the aqueduct suction main to supply critical station services such as motor cooling, bearing cooling and seal ring flushing systems. Whenever the aqueduct is taken out of service, the station services supply is re-valved to be supplied from the pump delivery main via a pressure reducing, service water control valve (see Figure 1).

2.2 Station Emergency Shutdown System

In order to protect the station against critical flooding events, the control system includes a ***"Station Shutdown"*** function. When this occurs all pumps are stopped and the Station Isolation Valve, River Suction Gate and the Aqueduct Isolation Valve immediately close to protect the station from external sources of stored energy. Critical faults that initiate a station shutdown include:

- Delivery main reverse flow
- Pump well flooded alarm
- 110 Volt DC system failure
- Pressing the manual "Station Shutdown" E-stop button.



<u>Figure 1:</u> ***Pump Station normal configuration (River Duty)***

2.3 Event 1: May 12-13

On May 12th the Maroondah aqueduct was dewatered to the Yarra River to enable structural repairs to take place upstream of the pump station. The aqueduct scour valve was opened to dewater the aqueduct and the suction main down to the same level as the aqueduct isolation valve. During this time the station was offline as the river flow was too low to allow pumping and the aqueduct supply was now unavailable. The following day river levels had risen sufficiently to allow river harvesting to resume. To facilitate this, the station services supply was transferred to the rising main and one of the main pumps was started on river duty.

2.4 Event 2: May 19

During the 6 days between May 13 to May 19 the remaining section of the aqueduct suction main gradually dewatered a further 17 meters to river level through, what was later discovered to be, a passing suction valve on one of the main pumps. With the aqueduct service water valve now closed, there was no means to ensure the suction main remained fully charged and there was no online pressure monitoring available to alert the operator that the main was now empty.

In the afternoon of May 19, operators shut down the pump station as river flows had once again declined to below minimum environmental levels. During the pump stopping sequence a minor "*reverse flow*" was detected in the rising main. This alarm initiated a ***"Station Shutdown"*** sequence which automatically closed the Station Isolation Valve, River Suction Gate and Aqueduct Isolation Valve. The reverse flow alarm during a pump stop sequence was not uncommon and was known to occur periodically.

As the pumping station was no longer required to operate it was left offline in "Shutdown" mode with the major isolation valves remaining closed (see Figure 2).

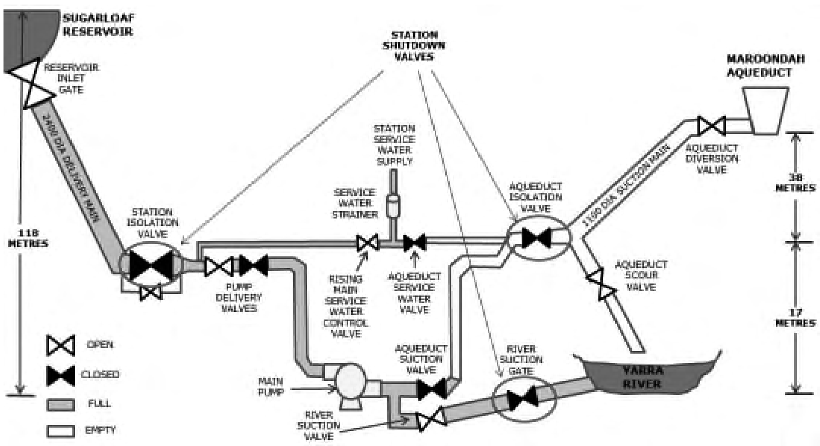


Figure 2: ***Station in "Emergency Shutdown" Mode with Aqueduct suction main dewatered***

2.5 Event 3: May 21

With the Aqueduct maintenance works now complete, the aqueduct is slowly recharged but only filled as far as the, now closed, aqueduct isolation valve leaving approximately 17 meters of empty 1100mm diameter suction main beneath it (see Figure 3).

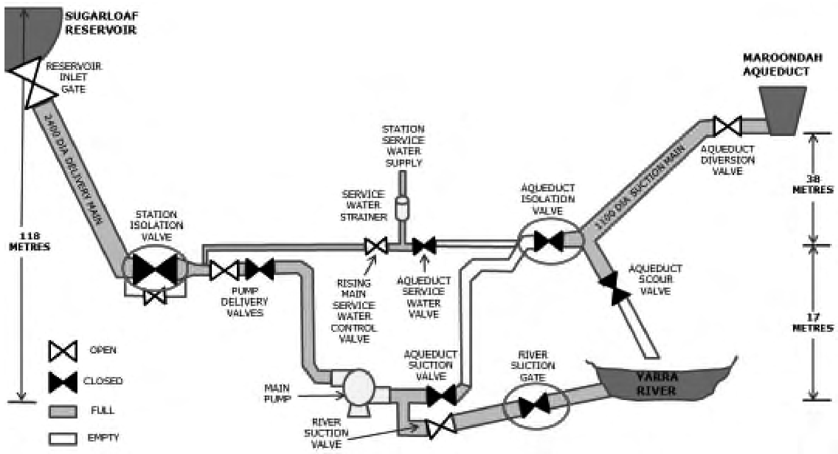


Figure 3: ***Aqueduct main recharged***

2.6 Event 4: May 22

The Operator attends site to prepare the station to resume pumping from the now recharged, aqueduct supply. He initiates a ***"Station Reset"*** command. This control function returns the Aqueduct Isolation Valve to its pre-shutdown condition, immediately opening the valve. The opening travel time of the 1100 mm diameter butterfly valve is less than three seconds and the rapid opening of this valve allowed the 40 tonnes of stored water above the valve to drop rapidly into the empty void below. The resulting pressure shock split the housing of the 250mm service water valve causing water to flood from the ground floor into the station below showering the high voltage pump drives with water and causing an electrical fault turning off all the station lighting.

The shock also hits the two closed, 1100mm diameter, aqueduct pump suction valves tearing the butterfly's from their cast housings and punching a 300mm hole in the side of one of the valves (Figure 4 & 5). Water flowing from these valves knocked down a blockwork wall and flooded into the pump well at a much greater rate than the sump pumps could manage.

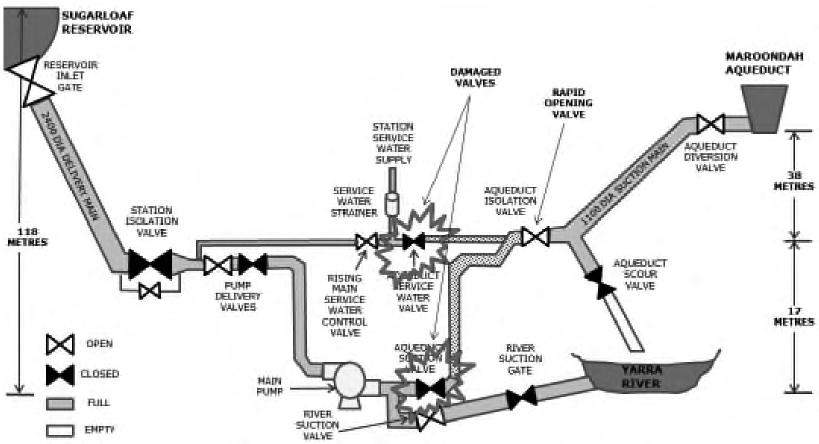


Figure 4: ***Location of damaged valves***

Figure 5: **D*amaged butterfly valves***

2.7 In the Line of Fire

The Operator Interface Unit used for station control is situated on "C" floor at the bottom of the pump well, one level above the main pumps. With water flooding over high voltage equipment from above, the pump well flooding from below and the station in total darkness, the operator was trapped in a precarious position. To add to the operator's predicament, an emergency escape ladder adjacent to the operator interface had been recently deemed non-compliant and was barricaded pending further investigation. This left the operator with no choice but to remain in this location (Figure 6) until help arrived to safely isolate the electrical supply.

2.8 Saved from Flooding

One of the alarm conditions that initiate the ***"Station Shutdown"*** sequence is a 110V DC supply failure. The water flooding in from the failed service water valve above faulted the 110V system initiating a Station Shutdown which closed the Aqueduct Isolation Valve preventing any further flooding. With the station power supply now faulted leaving the sump pumps no longer operational, the safety measure that initially led to this incident was the same measure that ultimately protected the station from being totally flooded.

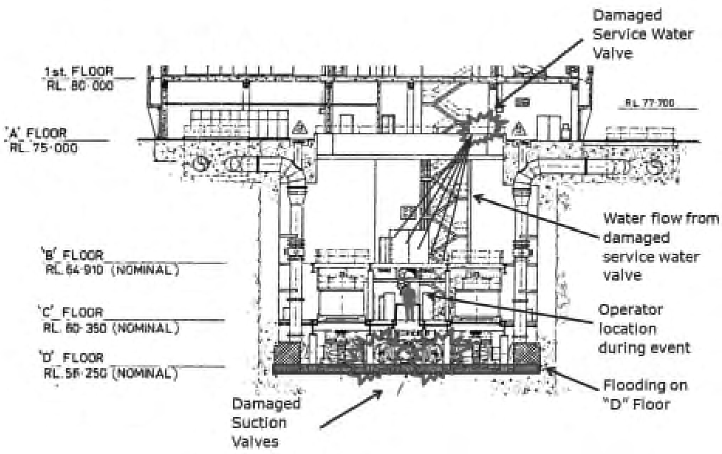


Figure 6: ***Operator location during the incident***

2.9 Recovery Works

With all personnel now safe and the station safely isolated, work began immediately to repair the station to an operational state. Transfer of water from the Maroondah Aqueduct was ceased and 10 Gigalitres of river water flowed past the station, unable to be pumped into Sugarloaf Reservoir. The Winneke Treatment Plant continued to supply water to Melbourne but, with no inflow, the level in Sugarloaf reservoir began to steadily decline.

A team of operations, asset management and project engineering representatives worked tirelessly together to implement the recovery project in as short a time as possible.

Orders were immediately placed for replacement valves. Meanwhile detailed isolation risk assessments and implementation plans were developed to allow the safe removal of the damaged valves and installation of blank flanges to enable the station to harvest from the Yarra River until the new valves arrived. A second shutdown was then conducted to remove the blank flanges and install the new valves returning the station to its full operating capacity. These restoration works were conducted over a period of eight months at a cost in excess of $1.2 million.

2.10 The Investigation

Following this incident a lengthy investigation was conducted using the "ICAM" investigation methodology. This investigation examined several key focus areas including:

- Organisational Factors
- Task / Environmental Factors
- Individual Actions
- Absent or Failed Defences

Whilst there were many key factors associated with this incident the root cause was ultimately determined as: ***"The hazard of the aqueduct suction pipe becoming dewatered was not identified in the design and subsequent operation of the pump station."***

2.11 Key Learnings

This incident involved hazards affecting the asset/technical integrity of the facility. The installed engineering controls did not effectively mitigate the process safety risks to asset integrity, which had the potential to affect personal safety.

In the facts and documentation considered during the investigation there was no evidence that the hazard of dewatering the lower section of the aqueduct suction pipe had been identified in the pump station design. The hazard was not identified in the relevant manuals, training or procedures, nor had it been observed in the operating experience of the pump station, and no mitigation measures were installed or implemented at the pump station. (Gall 2015)

The key learnings from this investigation include the importance of:

- Risk assessment processes during the design stage (HAZOP, CHAZOP & Safety in Design) to identify this type of hazard;
- Understanding hydraulic gradients and potential stored energy in pipes and pump stations;
- Monitoring and managing the differential pressure across control or reducing valves when de-isolating, recharging or resetting plant;
- Understanding how the loss of the control of stored energy can create process safety hazards, arising from the failure of asset/technical integrity, which have the potential to affect personal safety;
- Selecting the correct engineering controls to prevent the plant from operating in an unsafe condition e.g. bypass valves, control interlocks and permissive alarms;
- Ensuring asset risk assessment processes include consideration of process safety risks in evaluating the effectiveness of engineering controls and safe guards;
- Ensuring that risk assessments of operational or asset changes include a holistic (whole-of-system) review of the potential impact of the relevant change, and ensuring that those risk assessments are appropriately documented;
- Proactively reviewing local operations that may expose people to “line of fire” hazards in the event of an asset/technical integrity failure, including evaluating egress from these locations in the event of an emergency; and
- Ensuring core risks are identified for each operational site, and effective controls are in place and appropriately documented in procedures. (Gall 2015)

3.0 CONCLUSION

This incident was significant in identifying the need for renewed focus on process safety at Melbourne Water’s key high risk assets with potential to house large volumes of stored energy and high pressures. It also highlighted the potential for Operations and Maintenance staff to be in danger should a similar incident occur during routine plant operations. The ripple effect from this incident spread to all corners of Melbourne Water’s Service Delivery arm, as teams from both the water and sewer sides of the business took a fresh look at process safety at their keys assets and asked themselves the question, ***“Could something we don’t know be waiting to harm us on our site?”***

4.0 ACKNOWLEDGEMENTS

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- Aaron Ward: Process Engineer, Winneke Operations
- Peter Gall: Manager Eastern Treatment Plant

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FINDING THE MISSING FLOW ON LARGER PIPELINES

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42nd Annual WIOA
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FINDING THE MISSING FLOW ON LARGER PIPELINES

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ABSTRACT

An important percentage of unrecorded flow in any water network is through larger water meters referred to as Commercial & Industrial (C&I) meters, or bulk water meters. This is due to the inability of some C&I or bulk meters to register low flow rates.

While the amount of water measured through 20 mm domestic meters compared to that measured by larger bulk meters in the same fleet will vary from one council to another, it is almost always the case that the bulk users are some of the largest and highest value water customers of the council.

Capturing the most detailed and complete information from the water network is critical to understanding how and when the network is operating efficiently.

Being able to account for as much water as possible from the total amount of treated water leaving the water treatment plant is essential for both water balance reporting and revenue generation from the water being registered by water meters.

1.0 INTRODUCTION

In the case of bulk meters, under-registration can lead the water utility to underestimate its real losses, and its true level of non-revenue water.

Water meters are the cash registers, or some may say the 'cash cows' of councils and water utilities.

When a meter is under-registering, the council or utility is losing revenue.

Councils and utilities do have to accept that a portion of the water they treat and supply will not be accounted for. There are a number of reasons behind this. Losses can occur through line breaks, fire service testing, leaking infrastructure or even through incorrect meter reading. However, it is possible to minimise the amount of unregistered water. One difficult to manage source of loss has been water meters just not accurately registering the actual amount of water flowing through them.

The selection of the best water meter for the application is critically important as discrepancies in water registration across the meter fleet network can involve very large volumes of water and very large amounts of lost revenue.

In some utilities, unregistered water can be 15% and for a larger utility, this can mean millions of litres and many millions of dollars in lost or unregistered water.

One of the major challenges for utility and council water service providers is trying to accurately measure the total amount of water delivered. A significant portion of supplied water flows at rates lower than some traditional water meters are designed to measure. A challenge has been to design a cost effective product that can accurately measure the majority of this lost flow.

2.0 DISCUSSION

There should be a very high level of understanding within the water utility about initially selecting the correct meter size and type for every C&I water service application. This is the most essential component in ensuring the highest level of registration occurs throughout the meter fleet.

Occasionally the end service use at the same location may change. An example might be the cessation of most food manufacturing at let’s say the Golden Circle facility in Brisbane, the same facility is now used mostly for warehousing. In this example, there should be a reduction in the size of meter fitted at that service to best meet the new requirements, otherwise significant under-registration is likely to occur if the previous meter is left in place.

One typical bulk water service type for which it is very important to get the meter selection right, is any service that has a component of domestic usage within that service. A classic example is a block of housing units, where at peak hours of usage, flow rates can be high, but in off peak times, flow rates can be extremely low.

This is a service type that should have a meter fitted that is able to register those very low flow rates, as well as being able to register higher peak flow rates. Other examples in this type of domestic end use category can be hospitals, caravan parks and schools.

Let’s look at the main types of bulk meters being used and some of their features.

But before we do that, we need to be aware that one of the most important matters to be aware of with any bulk water meter is what is called the turn down ratio. This is the Q3/Q1 figure or also referred to as the R ratio. Generally the higher this turn down ratio number or R number is, the more capable the meter is of registering low flow rates.

Of course the meter must be reliable, have a good working life and be effectively priced.

Meters from 25-40 mm have generally been of the mechanical volumetric measurement type and these generally have a turn down ratio of R200 to R400. Good quality versions of this meter type are very robust, very reliable and very cost effective.

We have started to see some electronic meters including ultrasonic and mag flow being offered in these sizes. At this early stage they are quite expensive and have similar turn down ratios to the volumetric meters in these sizes. In some cases the R ratio claimed may be a bit higher than the mechanical meters but we will discuss some technical concerns, and limitations later.

There can be advantages in some electronic meters in comparison to mechanical meters when measuring raw water or water with contaminants. However, this paper is discussing potable water meters.

In meter sizes 50 mm and larger, the most common meter used has been the Helix meter. The Helix meter has been around in one form or other for more than 50 years, with various design and technical improvements over that period. The Helix meter has been the ‘go to’ meter for a majority of utilities. They have been extremely reliable and inexpensive.

However, they have relatively low R ratios. If the meter size used is too large for a particular service, which is a very common problem, then the standard Helix meter will not register all of the flow to the particular end use service.

There is a temptation to select a lower quality and cheaper version of Helix meter due to all Helix products being perceived as uniform, by some customers. We believe this is exacerbating the problem of under registration, as some poorer quality Helix meters display high accuracy degradation.

Good quality combination meters are without doubt the best meter available for capturing very low flows to high flows, and they have the highest R ratio of any commercially available water meter on the planet. An R 16,000 is possible. However, combination meters are heavy, have a higher initial cost and also have some ongoing maintenance costs particularly with replacement of the bypass meter required from time to time, to maintain the high turndown ration. It can be necessary to replace the bypass meter on a combination meter every year or two, depending on the application, because the bypass meter does so much work. If the bypass meter is not replaced when required, then the meter may degrade to an R ratio of the basic helix meter used as the main meter which may be less than R100.

Commercial single jet meters (CSJ) are available with a higher R ratio than for a Helix meter. CSJ meters have a better low flow capability for the same size meter as a Helix, but have a significantly lower maximum flow rate capability. This is due to the Helix meter having a balanced thrust rotor design, compared to a CSJ meter using an unbalanced thrust rotor design.

Ultrasonic meters in larger sizes have been available for a relatively short time on the market. They have up to an R 500 typically. It is uncertain at this stage if this meter type is going to be suitable for all applications or not.

Battery powered electromagnetic flow meters are more commonly called mag flow meters. These meters typically have R ratios of between 200 and 400. However, most battery powered mag flow meters use a very slow sampling rate of 15 seconds to conserve battery life. This means the meter only ‘samples’ the flow every 15 seconds. Particularly at low flow rates, this can lead to under registration, or in effect a lower actual R ratio than claimed. This feature is true to some extent with any fully electronic meter, and needs to be seen as a bit of a limitation.

As an example, comparative in field testing of one mag flow meter with a 15 second sampling rate, and one with a 0.5 sec sampling rate showed that the meter with the 0.5 sec sampling rate registered more than an additional 10% at low flow rates. The differential was less at higher flow rates, but clearly demonstrates this limitation of fully electronic meter design.

A relatively recent meter development is a hybrid Helix meter which has combined the robust capability of a helix meter, with an electronic pick up within the water meter.

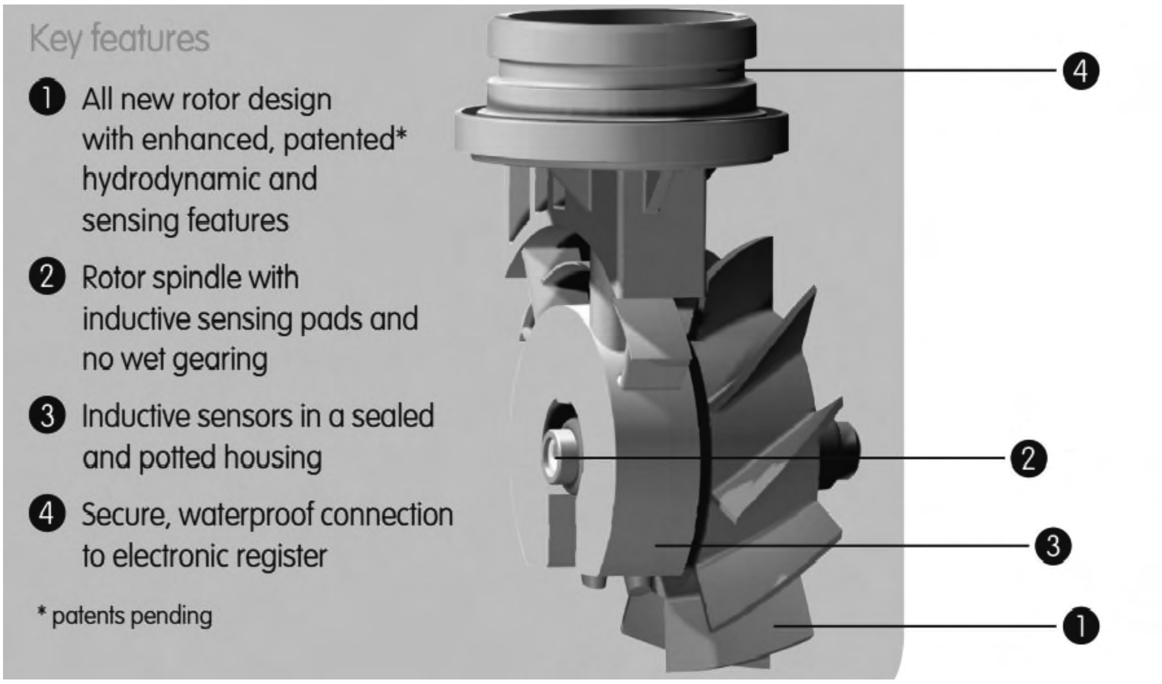

Figure 1: ***Hybrid Helix meter***

This has allowed the hybrid Helix meter to commence registering at significantly lower flow rates, compared to a conventional Helix meter. The H5000 hybrid Helix meter for example has been certified up to an R 2000, which is a very significant improvement.

Elster Product Comparison

DN50 specifications shown for comparison purposes

Product	H4000	S2000	Q4000 low-flow	C4000	H5000 (extended)
Technology	Woltmann	Single Jet	Electro-mag	Woltmann/Piston	Woltmann
Q3 Permanent Flow (m3/hr)	63	16	40	25	63
Q1 Minimum Flow (m3/hr)	0.63	0.10	0.10	0.016	0.05
Q3/Q1	R100	R160	R400	R1600	R1250
Installation	Any	Horizontal	Any	Any	Any
ΔP @16 m3/hr	0.015	0.18	0.005	0.35	0.015
Comms	PR7 module	PR7 module	MX39 module	PR6+PR7 modules	Integrated
Battery Life (module / meter)	Up to 10 yrs	Up to 10 yrs	10 yrs+	Up to 10 yrs	15 years typical[1]

1. 15 year life is in temperate climates with typical flow profile. Minimum battery life in all conditions is 10 years

18

Figure 2: ***50mm meter comparison – Elster***

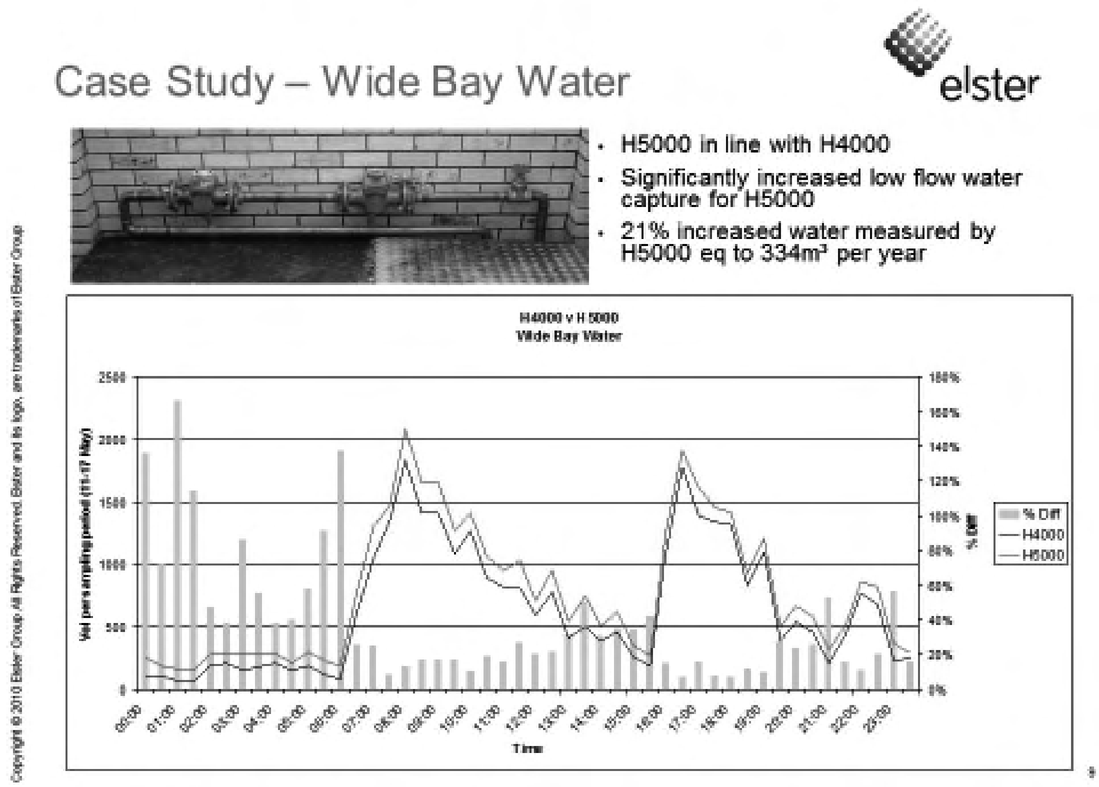

Figure 3: ***Case study from Wide Bay Water***

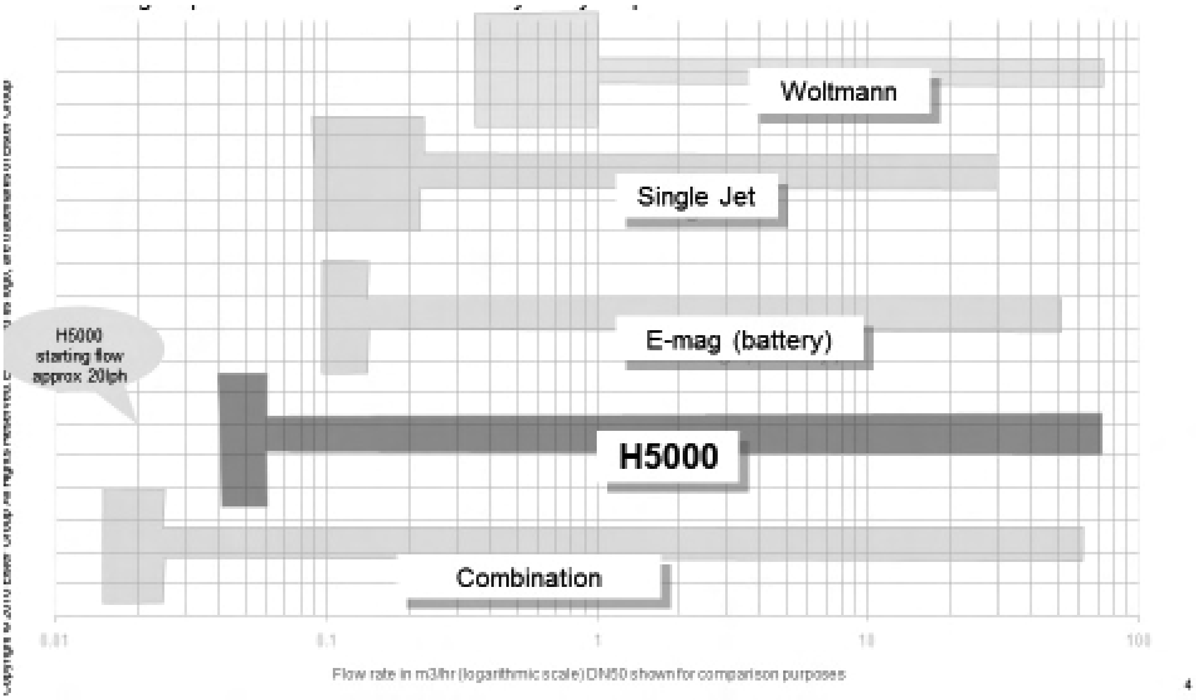

Figure 4: ***50mm meter accuracy envelope***

3.0 CONCLUSION

Low flow measurement and capture of total water usage is a challenge for councils and utilities. With an ever increasing focus on accounting for every drop of treated water, coupled with trying to reduce operational costs, the demand for a simple to use, simple to install and simple to maintain water meter product is high.

If changing to a higher specification and higher cost meter product, you need to be confident of a positive cost benefit analysis.

Some analysis has demonstrated, a pay-back period of less than 2 months is possible for some services if replacing a H 4000 for example with a hybrid H 5000.

It is important to remember that if we can’t measure water usage accurately, we can’t manage it properly.

THE ROTARY SCREEN THICKENER: INCREASING ANAEROBIC DIGESTION GAS PRODUCTION

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THE ROTARY SCREEN THICKENER: INCREASING ANAEROBIC DIGESTION GAS PRODUCTION

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ABSTRACT

Queensland Urban Utilities (QUU) has an extensive capital works program implemented to upgrade infrastructure at the Luggage Point Sewage Treatment Plant (STP). Luggage Point STP treats domestic and trade waste from approximately half the population of Brisbane, with a population of over 2.3 million people and growth rates forecast to rise. It is essential QUU, and stakeholders look at improving the Luggage Point STP process to cater to the population growth and ensure a sustainable solution is achieved for this catchment area.

The original plant was built in 1975, and has evolved as the population has increased. QUU have identified that the plant is in need of many upgrades to provide more efficient treatment of the sludge and effluent streams. The Rotary Screen Thickener (RSTs) is a process used to thicken primary sludge, in order to prepare the sludge for anaerobic digestion. The Luggage Point STP has 6 anaerobic digesters, producing and capturing methane gas (CH^4) which is then used to power the Cogeneration (COGEN) system. QUU implemented upgrades on the COGEN facility and the RSTs to help improve the sludge stream and move towards achieving the QUU Energy Strategy. This planned strategy aims to reduce energy costs, devise ways to improve energy consumption, energy generation and importantly, minimise greenhouse gas emissions. QUU has engaged Aquatec Maxcon (AQM) to replace the aged RST facility, which was nearing the end of economical service life, with an aim of improving the quality, operability and reliability of the RST system.

Figure 1: ***Luggage Point STP***

1.0 INTRODUCTION

The anaerobic digesters produce on average 18,000 m^3 of biogas a day. The digesters are heated to maintain a constant temperature in the mesophilic range of approximately 36^o Celsius. The digesters process Thickened Primary Sludge (TPS) from the RSTs and Thickened Waste Activated Sludge (TWAS) from the Dissolved Air Flotation system (DAFs).

The methane producing bacteria need essential conditions to thrive and several processes to contribute to the production of methane. In order for the digesters to operate efficiently, the operator needs to monitor the aforementioned processes. The operator faces many challenges and issues when it comes to the operation of the plant. These can include wear and tear, planned plant shut downs, planned and/or responsive maintenance, and plant loading.

1.1 Issues and Plant Monitoring

Primary Sedimentation Tanks

The Primary Sedimentation Tanks (PSTs) operate to allow the sludge to settle and be drawn off from the draw off pits. The sludge is collected using a mechanical bridge that pushes the sludge to the draw off pit. There are 6 PSTs and they are paired up to allow the sludge to be drawn off in 20 minute sequences. The sludge draw off target is 300-380 kL/d/per pair of PSTs at 1-2% sludge concentration. The primary sludge is then pumped to the RST feed tank for further thickening. The operator needs to monitor the draw offs from every cycle to ensure that the sludge is removed from the PSTs. PSTs sludge discharge pumps can become air locked causing them to fault and skip the draw off cycle resulting in sludge to build up in the PSTs.

The Rotary Screen Thickeners

The old RSTs had high levels of wear and tear due to the age of the plant, and corrosion caused by hydrogen sulphide. This results in process inefficiencies which were verified in the output data. Consequently the RST upgrade was approved and contracted to AQM in late 2016, and the system was successfully commissioned in April 2017.

Figure 2: ***The commissioned rotary screen thickeners by Aquatec Maxcon***

Dissolved Air Floatation

The DAFs produce a TWAS sludge concentration of 4-7% and can use 50-70 kg of polymer a day. Monitoring the poly dose rate is essential due to the change in the waste activated sludge flow rates.

The TWAS pumps run on a VSD and P&ID level control sensor. When the WAS tank level increases the pumps speeds up to increase the output. The operator monitors the subnatant quality and TWAS concentration to ensure optimum output.

Pump Pressures

The DAFs and RSTs sludge discharge systems are equipped with progressive cavity pumps operating on variable speed drives (VSDs) with a P&ID control utilising level sensor in the sludge hopper. When the hopper reaches a certain level, the VSD increases the pump speed to remove the sludge at a higher rate. The operator must monitor the discharge pressures of these pumps. The pumps will alarm at 400 kPa to indicate "high pressure" and interlock on "high high pressure" when the pressure exceeds 500 kPa. The alarm indicates that the TWAS and/or TPS is above its optimum thickness. So monitoring pump pressures is a good indication of overall DAFs and/or RSTs operation. This is a good indicator to identify whether the poly dose rate is adequate or a possible process upset has occurred.

Anaerobic Digesters

The anaerobic digesters use pumping system for recirculation. Ragging can be occasionally found in the digesters, and can block up the recirculation pumps. The ragging issue can also block up the suction lines of the heat exchangers causing issues with maintaining optimal operating temperature of the digesters.

2.0 DISCUSSION

The gas production relies on many aspects of the plant to work efficiently. The cooling water used for cooling the COGEN engine while in operation is used to maintain the temperature in the digesters at 36^{o}C through heat exchangers. If the COGEN is unavailable, a standby boiler is available to assist with heating the digesters. The ideal method of heating the digesters is to use the COGEN, as the boiler struggles to heat the volume of water essential for the digester operation, and in turn affects the digester efficiency.

The total solids content of the digester feed sludge affects the performance of the anaerobic digesters. In order to increase the digester efficiency it is important to understand the total solids concentration theory. At Oxley Creek STP operated by QUU there is a sludge process called the CAMBI (Thermal Hydrolysis). The CAMBI uses the sludge from the belt filter presses, and also the sludge from Fairfield, Wacol, Gibson Island, Wynnum, and Sandgate STPs. In theory the CAMBI is cooking the sludge and reducing the volume through heat and destruction of the cell structure. The sludge feed to the CAMBI on average is 14% DS solids concentration and is then reduced in volume by up to 3 times to 5 to 8% DS. This has proven an increase in gas production (56% methane), better performing digesters and better quality cake solids of 22-27% produced from the centrifuges.

At Luggage Point STP the main control to ensure the digesters operate efficiently is through optimising the sludge stream processes and ensuring each process operates according to the operating envelope. This includes the PSTs, RSTs, and DAFs processes. The COGEN in operation helps accomplish the heat needed to reach optimum efficiency out of the digesters.

2.1 Sludge Stream Optimisation

After an investigation, it was found that DAF 1 & 2 were being over-dosed with polymer. Therefore, an analysis of DAF 1 & 2 was completed to identify ways to achieve the appropriate sludge concentration and to minimise the polymer dosing.

Table 1: ***The polymer dose rates and sludge concentrations***

	DAF 1 & 2	**DAF 3 & 4**	**RST**
Feed Solids (% DS)	1.00	0.65	2.40
Sludge Feed Rate (L/sec)	33.0	14.5	13.0
Polymer Dose Rate (kg/t DS)	1.2	1.1	2.3
Polymer Solution Strength (%)	0.12	0.11	0.20
Poly Dose Rate (L/min)	20.4	11.4	16.8

Tests were completed and trials were undertaken to identify how much reduction of the polymer dose rate was needed to achieve 7% dry solids.

Table 2: ***Optimising the DAF test results***

Trial No.	**Polymer Dose Rate (L/min)**	**DAF 1 Thickened Sludge (% DS)**	**DAF 2 Thickened Sludge (% DS)**	**Total Polymer Use Per Day**	**Savings Of Polymer Use Per Day**	**Savings Per year**
1	20.4	7.80	8.16			
2	17.0	7.55	7.40	4,896 litres per day		
3	15.0	7.40	7.52	2,880 litres per day	7,776 litres per day	$22,384 .00

2.2 DAF Investigation

It was found DAF 1 & 2 was dosing more polymer then required so a reduction of polymer dose was trialled. Sludge sample 1 was taken before the change at 20.4 L/min polymer dose rate to identify the solids concentration and two changes were made in the polymer dose rate. The first change was at 17 L/min and then the second change at 15.0 l/min. This achieved a reduction in solids concentration while maintaining over the 7% DS requirement. This reduction saved 7,776 litres of polymer dosed at 0.12 % poly concentration. This saving is estimated to $14,284 per annum on polymer costs. There is also potable water savings of approximately 2.7 ML per year which is estimated to be $8,100 at $3.00 kL. This gives a total saving of $22,384 per year.
Note: neglecting pumping energy savings

2.3 RST Optimisation

The RST is in the proving period being completed by Aquatec Maxcon and samples have been collected to test the new RST's. Due to the recent start up at the time this paper was being compiled, a limited number of sampling results were available to the staff. However, the system is displaying encouraging performance.

Table 3: ***The old RST grab sample results from SAS Laboratory***

	03/03/2017	**17/03/2017**	**07/04/2017**
Feed Solids Concentration (% DS)	2.10	2.70	2.20
Filtrate TSS (mg/L)	11,680	15,960	15,380
TPS (% DS)	4.0	3.6	6.0
Capture Rate (%)	44	41	30

Table 4: ***The new RST grab sample results collected by an operator***

	RST 1	RST 2	RST 3
Feed Solids Concentration (% DS)	2.40 to 3.10	2.40 to 3.10	2.40 to 3.20
Polymer Dose Rate (kg/t DS)	2.3	2.3	2.3
TPS (% DS)	6.9 to 7.8	8.0	7.2 to 7.4
Filtrate TSS (mg/L)	560 to 1,850	420 to 2,450	520 to 1,600
Capture Rate (%)	94 to 98	92 to 98	95 to 98

The new RSTs had a significant improvement in sludge and filtrate quality and system capture results.

2.4 QUU Energy Strategy

The biogas is extracted from the digesters to use as an energy source to fuel the COGEN. The COGEN at Luggage Point STP is operated to provide 5 kW/sec of energy towards the operation of the plant. There are 2 Jenbacher engines (1.1 Megawatts each) onsite functioning in a Duty/Standby mode. Industry experience indicates >250 kW (100 m^3 biogas/hr), and at Luggage Point STP the excess biogas is burnt off using the flare. Energy consumption at STP's is primarily driven by aeration (50% +), Bioreactors (10% +) and pumping (10% +). Finding ways to optimise the plant and reduce energy costs are essential to achieving the QUU Energy Strategy.

The COGEN has a gas scrubber that helps reduce the H_2S in the biogas and clean the gas to help the COGEN reach its maximum efficiency. The engines can ramp up to 1,121 kW to provide the plant with the energy required. It was identified that it was best to run 1 engine at full load rather than 2 engines at partial load, based on the analysis outlined in Table 5.

Table 5: ***Costing involved in operating 1 engine at full load or 2 engines at partial load***

	Energy Generation Savings ($/year)	Maintenance Cost ($/year)	Net Savings ($/year)
Operating Scenario 1 – 1 engine at full load	941,875	380,000*	561,875
Operating Scenario 2 – 2 engines at partial load	1,093,248	600,000^	493,248

<u>Figure 3:</u> ***The COGEN Plant at Luggage Point STP***

3.0 CONCLUSION

QUU is dedicated to improving and optimising the Luggage Point STP. The digesters are critical plant that can help provide financial benefits for QUU by managing energy related costs and minimise greenhouse gas emissions. In order to maintain the QUU Energy Strategy at the Luggage Point STP the operators have identified that closer monitoring of the sludge stream processes will help provide better quality sludge feed to the digesters. This will increase gas production and improve the sludge solids concentration pumped to the centrifuges. It was proven at Luggage Point STP that a 1% increase in centrifuge cake dry solids was a significant savings of $80,000 a year in cartage costs. Furthermore, the anaerobic digester gas production, since the start of new RSTs system operation, has significantly improved from averaged 2,900 m^3 of methane gas to current production of 3,500 m^3.

Significant savings can also be created by optimizing the DAF and RST by monitoring process samples daily and managing the polymer dose rates accordingly. The replacement of the RSTs by Aquatec Maxcon has shown encouraging improving to the system operation, in both thickened sludge quality and improved capture rate by the RSTs. This will help maintain the digester sludge feed rate of 6%-9% and preventing any over dosing of polymer. Through an investigation an estimated savings of $14,284 was found in chemical costs, and a saving of 2.7 ML ($8,100) of water per year by optimising the DAF plant. These changes can potentially save QUU $22,384 per annum.

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UNDERSTANDING BACKFLOW PREVENTION – THE THINGS YOU NEED TO KNOW

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UNDERSTANDING BACKFLOW PREVENTION – THE THINGS YOU NEED TO KNOW

Peter McLennan, *President*, The Backflow Prevention Association of Australia Inc.

ABSTRACT

Design, installation and hydraulics of backflow prevention devices. Understanding the pressure drop and flow restrictions.

Backflow prevention devices have become an integral part of the water supply asset. With the protection of the drinking water being paramount, backflow prevention is required by the Plumbing Code at any connection that is subject to being affected by a cross connection. The operation and subsequent restraints backflow prevention devices present is not well understood. This paper discusses the design requirements and the performance characteristics of Reduced Pressure Zone backflow prevention devices so that water supply personnel can be cognizant with potential issues before they occur.

Topics covered include:

- Backflow Basics
- Pressure drop Vs flow rate and the issues
- Installation guidelines
- Discharge from Reduced Pressure Zone valves

Backflow prevention devices are like smoke detectors. They both save lives. Ignorance in understanding how they operate within your network is not an excuse when the coroner knocks at your door.

1.0 INTRODUCTION

Backflow is the term used to describe the reversal of flow in a water supply pipe or system. Backflow prevention devices are used to protect drinking water from contamination where cross connections occur.

The Australian Plumbing Code Section 4 relates to Cross Connection Control and covers the design and use of backflow prevention devices. Australian State and Territory Governments reference the Australian Plumbing Code in their regulations therefore mandating the management and use of backflow prevention devices.

Since the introduction of backflow prevention devices to the plumbing code in 1998, it is estimated that there are more than 500,000 testable backflow prevention devices installed across Australia. Other than a few in the plumbing industry, many people responsible for managing and maintaining drinking water systems know little of backflow and how the devices affect the system.

This paper attempts to demystify backflow and presents the basics so that even non-plumbing people can understand how they interact with the water supply.

2.0 DISCUSSION

Whether a plumber, hydraulic designer, plumbing consultant, water officer, water engineer, contractor or property owner, there are several things you must understand when considering installing or requiring the installation of a backflow prevention device.

A backflow prevention device is a safety valve that protects the drinking water supply. They are used extensively but many people don't understand the operation or the limitations and constraints applicable to these devices.

The following topics need to be addressed when considering the use of a Reduced Pressure Zone Valve (RPZ) backflow prevention device.

- Backflow Basics
- Pressure Drop
- Flow rates
- Discharge from RPZs
- Installation Guidelines

2.1 Backflow Basics

The Plumbing Code:

AS/NZS3500.1-2015 is referenced in the National Construction Code, Volume 3, the Plumbing Code of Australia. Section 4 is Cross Connection Control and lays out the backflow prevention requirements mandated through legislation. When a State or Territory references the National Construction Code in their plumbing regulations, unless otherwise stipulated, it makes the use of backflow prevention devices mandatory.

The Standard:

Backflow prevention devices are Watermarked to AS/NZS2845.1-2010 or AS/NZS2845.1-1998. The Watermark is your assurance that the device has been manufactured and tested in accordance with the relevant Standard. If it does not have a Watermark it should not be installed in the drinking water network.

Hazard Ratings:

The Standard identifies 3 levels of hazard (the contamination or pollutant that can come in contact with the drinking water)

High Hazard – The pollutant or contaminant if ingested could kill you. Facilities connected to the water supply likely to have this level of potential contamination would include mineral processing, meat processing plants, hospitals, mortuaries, plating works, etc.

Medium – The pollutant or contaminant if ingested is unpleasant and may make you ill. Facilities connected to the water supply likely to have this level of potential contamination include commercial buildings, schools, public parks, food processing plants etc.

Low – The pollutant or contaminant is non-toxic but is objectionable and should not be present in drinking water. Facilities connected to the water supply likely to have this level of contamination include residential homes, rainwater tanks etc.

Two types of cross connections:

1- A direct connection. This is where the cross connection is 'hard piped' and is often installed by people unaware of the possible consequences. It could be a bypass line or a submerged tank filling connection.

2- An in-direct connection. The most common cross connection is a hose. A hose is an in-direct connection as the outlet can be used and left in all sorts of situations. For example, drain cleaning, chemical mixing, pipe flushing, pool filling etc.

Two types of backflow:

Backsiphonage – the pressure in the supply line is reversed causing the water to be sucked or run backwards. This is usually caused by a water main break in the street but can be caused by mechanical devices that rely on venturi action to draw water from the supply line.

Backpressure – the water pressure within the facility is greater than the supply pressure. Causes can include high head pressure found in high rise building and at the top of hills and mechanical equipment failures.

Two types of backflow preventer:

1-Testable – for use in high, medium or low hazard applications

2- Non- testable – for use in low hazard applications

Testable backflow prevention devices are designed to be able to be tested for operation effectiveness and for maintenance whilst installed inline and are suitable for use in high, medium or low hazard applications.

Non-testable devices need to be removed from line for maintenance and testing hence they are only suitable for installation in low hazard applications (see Table 1 for a list of common devices and the applicable hazard ratings).

Table 1: ***Common devices and the applicable hazard ratings***

Reduced Pressure Zone Device (RPZ)	Testable	High hazard applications
Double Check Valve (DCV)	Testable	Medium hazard applications
Pressure Type Vacuum Breaker (PVB)	Testable	Low hazard applications
Dual Check Valve (DUCV)	Non-Testable	Low hazard applications
Vented Dual Check Valve (DCAP)	Non-Testable	Low hazard applications
Atmospheric Vacuum Breaker (AVB)	Non-Testable	Low hazard applications
Hose Connection Vacuum Breaker (HCVB)	Non-Testable	Low hazard applications
Single Check Valve Testable (SCVT)	Testable	Medium hazard applications only on fire lines

2.2 Pressure Drop & Flow Rates

Backflow prevention devices rely upon pressure drop across the check valves for effective operation. The minimum spring differentials are stipulated in the Standard and all Watermarked devices must comply.

Testable backflow prevention devices are field tested for effective operation upon commissioning and at least annually by an accredited tester trained in backflow prevention.

Always check the manufacturers published literature for the pressure drop curve to ensure you have enough available pressure to supply the amount of water required. It is especially important where a fire connection is concerned.

The following examples are from manufacturers published literature and should be used as a minimum.

- 100mm Reduced Pressure Zone Valve at 20 L/s has a head loss of 68 kPa *(Apollo Valves All Valve Industries product catalogue page 7)*

- 100mm Double Check Valve at 20 L/s has a head loss of 20 kPa *(Apollo Valves All Valve Industries product catalogue page 6)*

- 100mm Double Detector Check Valve at 20 L/s has a head loss of 68 kPa *(Apollo Valves All Valve Industries product catalogue page 8)*

- 100 Single Check Valve Detector Testable at 20 L/s has a head loss of 57 kPa *(Pentair Valcheq backflow prevention catalogue page 4)*

These figures are devices only and do not include strainers or isolating valves. These values must be considered where pressure is limited. It is not unusual for a complete assembly comprising of isolating valves, strainer and the RPZ having a pressure drop close to 100 kPa.

2.3 Discharge From RPZ Valves

All RPZ backflow prevention devices will dump water through the vent in the valve. It is a safety feature that ensures that if the device fails or there is a backflow event, the drinking water is protected.

The spillage of water is often inconvenient, but when installed where it cannot get away, it can become dangerous to property and humans.

All manufacturers publish the discharge rates applicable to their devices so be aware of these when you install an RPZ.

Otherwise you may have a flood on your hands as for example, a 50mm RPZ with a pressure of 700 kPa can discharge around 660 l/min, sufficient water to empty an Olympic sized swimming pool in 6 hours. See figure 1.

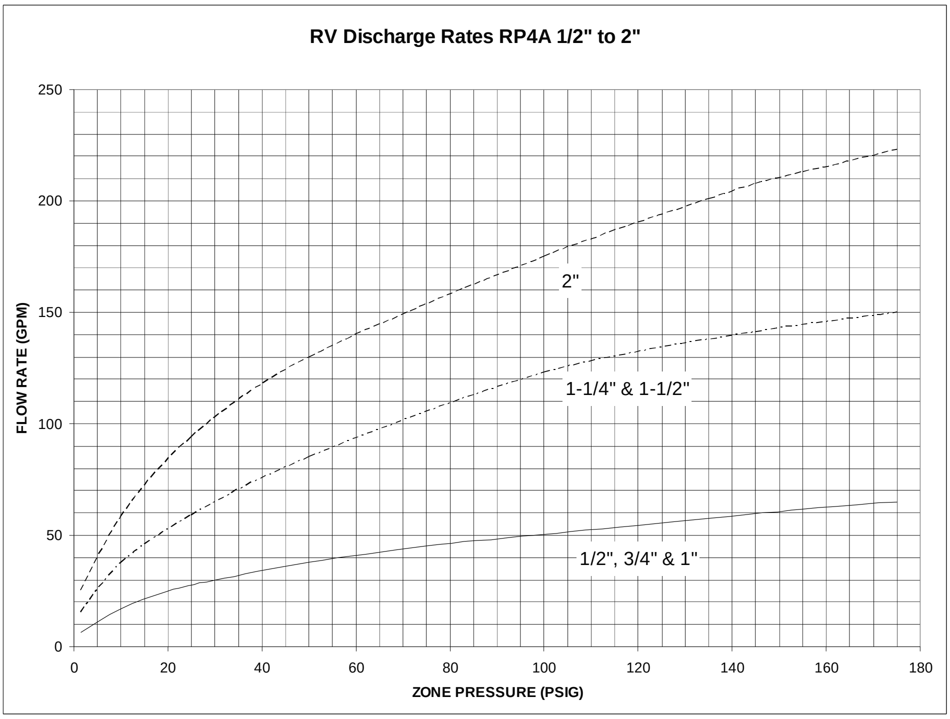

Figure 1: *Apollo Valves RPZ discharge Rates*

3.0 INSTALLATION GUIDELINES

Backflow prevention devices are mechanical devices that require regular testing and maintenance. To facilitate this, they must be installed where ease of access is available. The Standard addresses some aspects, but each manufacturer outlines specific installation requirements in their published literature.

The three questions you need to ask yourself are:

1 Is the device I am installing suitable for vertical and horizontal installation or just horizontal?
Reduced Pressure Zone backflow prevention devices are designed to discharge water either during pressure fluctuations or mechanical failure. To not compromise the level of safety, they are to be only installed in the horizontal plane. There is no Watermarked RPZ device approved for vertical installation.

2 Is the device I am installing suitable for concealing in a valve box or pit?
Due to the discharging of water, a valve box is susceptible to flooding. Once the water level covers the discharge vent, the valve is compromised and the safety reduced.

3 Does the device I am installing have ease of access for regular testing and maintenance without the need for special equipment or dismantling from the line?
AS/NZS2845.1 2010 stipulates that testable backflow prevention devices are to be commissioned upon installation and tested at least annually to ensure effective operation. Work place health and safety guidelines would dictate that backflow prevention devices not be installed in confined spaces, near hazards, in elevated positions or in ceiling cavities.

4.0 CONCLUSION

It is an everyday occurrence where the installation of a backflow prevention device is utilised to protect the drinking water, but to other than a few trained professionals in the industry backflow prevention devices remain confusing and are not understood by the very people responsible for providing the safe drinking water.

With the Australian Plumbing Code being referenced in most State and Territory legislation, it is mandatory for backflow prevention devices to be installed to protect the drinking water.

Understanding what backflow is and the limitations of the various devices will not only allow you to satisfy your duty of care where these devices are used, but to be able to understand the impacts the installation of these have within your network.

Backflow prevention devices are like smoke detectors. They both save lives.

5.0 ACKNOWLEDGEMENTS

Thanks to Apollo Valves / Conbraco Industries and the numerous 'unsung heroes', the dedicated backflow professionals who have inspired me to develop this presentation.

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MANAGING THE SUNSHINE COAST COUNCIL BACKFLOW REGISTER

Paper Presented by:

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42nd Annual WIOA
Queensland Water Industry Operations Conference and Exhibition
Logan Metro Sports Centre,
Crestmead
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MANAGING THE SUNSHINE COAST COUNCIL BACKFLOW REGISTER

Morgan Way, *Plumbing Inspector (Backflow Officer)*, Sunshine Coast Council

ABSTRACT

Taking Clean Water for Granted...

Every time you fill a glass with water from the tap, prepare a meal, or take a bath, you take for granted that the water will always be clean, pure, and safe.

Occasionally, situations occur outside of a council's control that can jeopardise the quality of the drinking water. A very common occurrence in a local water network operating system is the temporary loss of pressure due to the breakage of a water supply pipe or water main.

When these situations occur, conditions are present that can allow the backflow of pollutants or contaminants into the water system and threaten the purity of the drinking water system.

Sunshine Coast Council has a progressive policy developed and implemented to manage the design, installation, and repair and testing of backflow devices in the Sunshine Coast Council area of operation.

Backflow is a term in plumbing for an unwanted flow of water in the reverse direction.

1.0 INTRODUCTION

The Sunshine Coast Council area is located in south-east Queensland, about 100 kilometres north of Brisbane.

Sunshine Coast Council oversees the supply of clean, safe drinking water delivered from the Network Operator (UnityWater) in a land area of 229,072 hectares. The Sunshine Coast as of 2016 census has a population of 292,990 an currently have registered 11,000 devices.

Sunshine Coast Council has a statutory obligation to maintain a register for the maintenance and testing of backflow prevention devices installed in the region.

Sunshine Coast Council is the regulator for plumbing and drainage within its local authority boundaries. The Council is responsible for inspecting all plumbing and drainage work in the area of operation. This area ranges from Peregian Beach in the north, down to Coochin Creek in the south. Backflow prevention has always been a priority for Sunshine Coast Council, to maintain the integrity of the drinking water supply and enforcement of non-complaint work as to the Plumbing Code of Australia (PCA), the Plumbing and Drainage Act (PDA), and Standard Plumbing and Drainage Regulation (SPDR) Queensland Development Code (QDC), and the Queensland Waste Water Code (QPWC). Sunshine Coast Council's dedicated backflow prevention program began in 2008. However, the backflow program had been in operation long before then, when the local authority came under the governance of the Caloundra City Council.

The Caloundra City Council's aim back then was, and still is, to identify companies with existing and potential non-conforming, hazardous backflow prevention devices as well as properties needing devices to be installed to prevent cross connections within their properties. The program has continued to evolve.

The importance of backflow Containment, Zone and Individual protection has become more relevant as regulatory requirements have changed, with recognised on-site cross connection risks from alternative water supplies, chemical production, trade waste operations, burst water mains and irrigation systems. Just to name a few.

The Sunshine Coast Council Backflow Management Strategy recognises these risks and has developed objectives, operating principles, and an evolving management system informed by consultation and improvements in technology.

Backflow Management Objectives

The Sunshine Coast Councils backflow management strategy is designed to protect clients, assets and infrastructure by managing and controlling risks caused by potentially harmful contaminants back-flowing into the water mains from clients' private properties.

There are several objectives:

- Register backflow Containment/Zone and Individual devices that have been installed in the Sunshine Coast Council area of operations;
- Build a list of accredited plumbers available to test devices and manage this data using a system that will automatically send letters and notices to customers; and
- Offer assistance to endorsed plumbers and home owners by way of education and guidance on backflow.

2.0 OPERATIONAL PRINCIPLES

Sunshine Coast Council will:

- identify properties that pose a backflow risk to the drinking water supply including Sunshine Coast Council assets - for example hospitals, refuse areas, commercial and industrial properties as well as class 1 buildings;
- consult clients throughout the implementation process, demonstrating due diligence and duty of care to clients;

- implement and maintain a cost-effective management system, including a database, reporting system and management control procedures; and
- promptly identify, record, report and rectify non-compliant properties.

2.1 Implications, if not implemented

- Sunshine Coast Council would not demonstrate the expected level of due diligence;
- Sunshine Coast Council clients could be exposed to contaminated water sources from cross connections from properties;
- Loss of history of installed or removed devices;
- Loss of compliance or installation issues; and
- Loss of income to manage an efficient system.

2.2 Benefits

- Minimisation of risk to clients;
- Due diligence requirements met;
- Active regulatory role within customers' properties;
- No contamination of drinking water from backflow; and
- Minimising administration costs to Sunshine Coast Council.

3.0 CONSULTATION

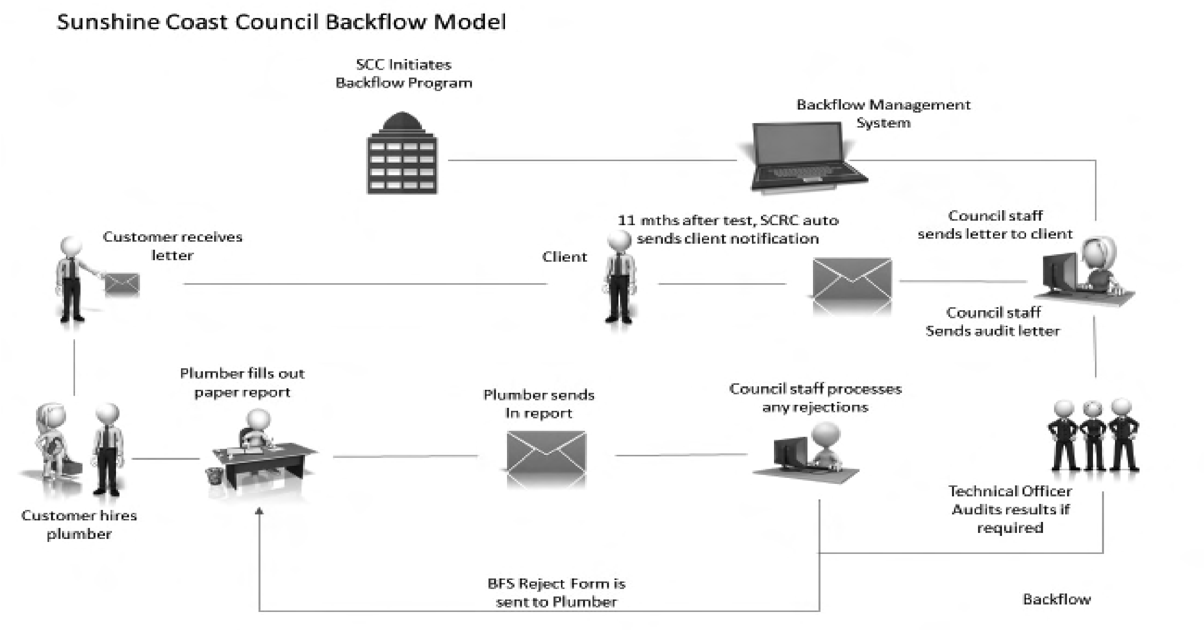
Sunshine Coast Council has consulted with clients throughout its operating areas as implementation will have significant impact throughout the region. The consultation included discussions with relevant industry groups, associations including hydraulic engineers, major and minor plumbing businesses.

Some areas of consultation included meetings with:

- The Endorsed Plumbers (Testers) to discuss their concerns;
- Maintenance staff of commercial and industrial sites to discuss concerns regarding installation and maintenance of their registers;
- Home owners to engage them in the correct maintenance program for the testable devices;
- Plumbing suppliers so they are informed on the required information that is prevalent to their areas of trade; and
- Fellow Council Plumbing Inspectors to discuss changes to these processes so they are engaged and informed of the requirements.

4.0 PAPER-BASED BACKFLOW MANAGEMENT SYSTEM

From 2008 to 2017, the Sunshine Coast Council has been reliant on Form 9's being delivered via Email, Post or Fax and even over the counter, for processing into the management system. We are moving progressively toward a fully computerised system via an App-based product predicted to be finished in the near future.



4.1 Issues from Paper Base Systems

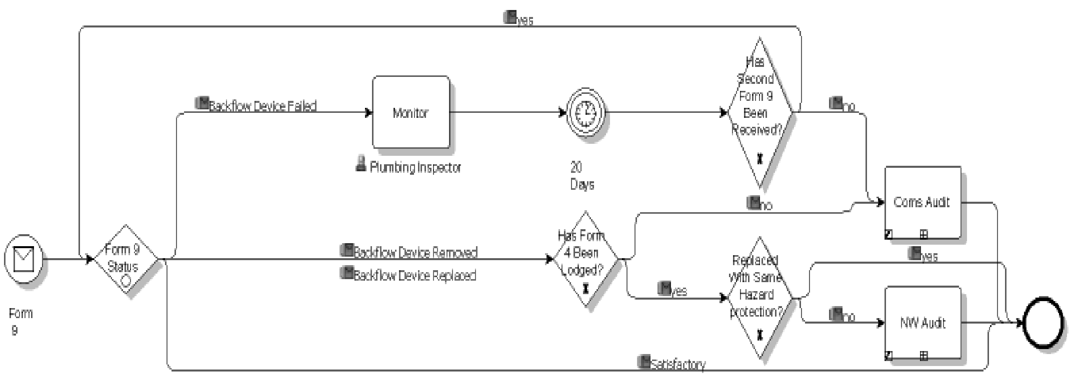
- Identification of properties requiring backflow prevention;
- Sending correspondence to clients who are required to install devices;
- Receiving Form 9's from plumbers that may have incorrect information and require correction;
- Entering the Form 9 information into a parent files and if required, its children;
- Deciphering the incoming data and ensure it is legible; and
- Raising an audit if the Form 9 has not met the full requirements of a desk top audit.

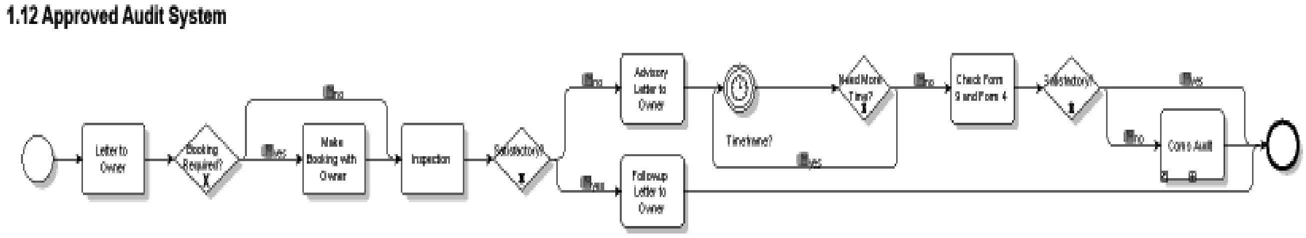
4.2 Designing and Developing the Electronic Data Base

The process of developing a database design consists of a number of procedures which are implemented by the database designer. Usually, the designer must:

- Determine the data to be stored in the database;
- Determine the relationships between the different data elements; and
- Superimpose a logical structure upon the data on the basis of these relationships.

1.10 Backflow Audit





4.3 Moving Towards a Fully Functional Online Lodgement System Our Vision

- From 2008 to now, Sunshine Coast Council is and will continue to allow plumbers to lodge test reports to the already existing management system; whilst continuing to allow paper based lodgement;
- Sunshine Coast Council is moving toward a fully online lodgement system by approximately 2018, rejecting paper-based lodgement except in exceptional circumstances, for example no computer;
- Endorsed Plumbers could lodge and store test reports for containment, zone and individual devices into the system;
- Endorsed Plumbers could retrieve all reports lodged under their name;
- All annual requested retest, installation and non-compliance letters would be sent through the online data base;
- Sunshine Coast Council Officers would continue with industry education by attending Registered Training Organisation backflow accreditation classes, Backflow Prevention Association Australia (BPAA) meetings and other plumbing industry groups meetings; and
- Monthly audits conducted on companies and Endorsed Plumbers would be, as it is now, a key function of the Sunshine Coast Council Backflow Management Program.

4.4 Advantages of Online Lodgement

- Free and straight forward process;
- Time saving, 24/7 lodgement of Form 9 and received immediately;
- Form 9 can be completed and lodged on site;
- Compatibility with desktop browsers (Firefox, Safari) and iPad Safari (additional mobile device compatibility);
- Reliability of information received by the Sunshine Coast Council;

- Less chance of inaccurate data or lost information;
- Benefit the customer in meeting their expectations; and
- Reduce the time needed to manage paper-based exceptions in the backflow system.

5.0 CONCLUSION

The backflow database is an essential management system which allows Sunshine Coast Council to accurately and reliably manage incoming and outgoing data.

Sunshine Coast Council are continuously improving systems to benefit endorsed plumbers and to ensure updated property information is available with accurate device information, changes to serial numbers as well as changes to locations if required.

Overall efficiency benefits will reduce manual adjustment errors in the data base. The six-month consultation period with relevant industry groups and associations, including hydraulic engineers, major and minor plumbing businesses will ensure that the system will be efficient and effective once implemented.

6.0 ACKNOWLEDGEMENTS

Licensed plumbers and endorsed plumber working in the backflow prevention industry.

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WHAT IS BACKFLOW & QLD BACKFLOW PREVENTION LOCAL GOVERNMENT MANAGEMENT REQUIREMENTS

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42nd Annual WIOA
Queensland Water Industry Operations Conference and Exhibition
Logan Metro Sports Centre,
Crestmead
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WHAT IS BACKFLOW & QLD BACKFLOW PREVENTION LOCAL GOVERNMENT MANAGEMENT REQUIREMENTS

Stephen Jennison – *President Backflow Prevention Association QLD Chapter*, Moreton Bay Regional Council

PRESENTATION

- Queensland laws to protect the water supply;
- What is backflow prevention;
- Why do we need a backflow prevention device;
- Authorised testers' requirements;
- Local Government licensing and registration of backflow prevention devices.

1.0 INTRODUCTION

When the *Sewerage and Water Supply Act 1949* (Qld) was amended in April 1992 it made way for Backflow prevention to be written into law in Queensland for the first time.

Having previously worked in the private sector, specialising in the testing and certification of testable backflow prevention devices, I joined the Pine Rivers Shire Council (now Moreton Bay Regional Council) as a plumbing inspector in 2005 and later became President of the *Backflow Prevention Association QLD Chapter* in 2014.

In 2008, the amalgamation of Pine Rivers Shire Council, Caboolture and Redcliffe Councils saw the Moreton Bay Regional Council established, creating the third largest Council in Australia with an area of 2,037 km2. **Moreton Bay Regional Council** is now one of the fastest developing regions in Australia, situated between Brisbane and the Sunshine Coast, and is currently home to 8600 registered testable backflow prevention devices.

2.0 DISCUSSION

This presentation is based on backflow prevention requirements in Queensland:

1. Plumbing and Drainage Act 2002;
2. Standard Plumbing and Drainage Regulation 2003;
3. National Construction Code Volume 3;
4. AS/NZS3500:2015 (Individual, Zone and Containment protection);
5. Submitting the testing reports with the Local Government;
6. Local Government creating a backflow licence and register.

3.0 ACKNOWLEDGEMENTS

I would like to thank George Wall and the entire conference organisation team for allowing me to speak on backflow prevention and provide a brief overview on how, why and where backflow prevention is required.

A WATER TREATMENT OPERATOR IN ANTARCTICA

Paper Presented by:

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A WATER TREATMENT OPERATOR IN ANTARCTICA

Gavin Allen, *Operator-Maintainer*, Veolia-ANZ

ABSTRACT

Veolia is collaborating on research into water and waste treatment technologies, under a five-year Memorandum of Understanding with the University of Melbourne (MoU, 2015-2020). The first project focused on sites at Australia's Casey Station, operated by the Australian Antarctic Division (AAD), in East Antarctica. This project aims to utilise low-energy biological processes, such as biofiltration, to treat summer melt water contaminated with diesel-based hydrocarbons.

Veolia's technical experts and operational staff have had the opportunity to participate in a water treatment plant development project at Casey Station, Antarctica. Under the direction of University of Melbourne Project Team Leader, Kathryn Mumford, and Australian Antarctic Division (AAD) Project Manager, Tim Spedding, the team were commissioning and operating a plant to treat water as part of the clean-up of contaminated sites impacted by diesel spills in Antarctica.

This paper will describe the process of retrofitting and re-commissioning a package water treatment plant designed to operate in the extreme Antarctic environment.

1.0 INTRODUCTION

A mobile water treatment plant (WTP) was constructed in 2002 for use at remote Antarctic contaminated sites during clean-up activities. The WTP features coagulation/flocculation, lamellar settling, bag and media filtration and finally ion exchange. The plant was designed originally to treat suspended solids and heavy metals from landfill leachate at Casey station in Antarctica. This WTP has been the subject of an intensive research program in remote cold regions site remediation since 2002 (See Figure 1).

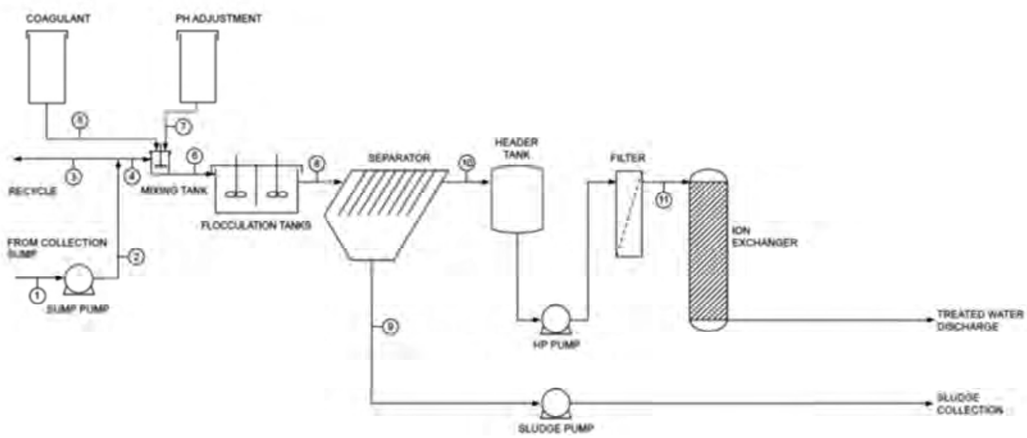

Figure 1: ***Process flow diagram of original process configuration of the Casey WTP***

More recently a new project has launched, this time associated with treatment of water contaminated as a result of diesel spills. Once again the WTP will be an important component of contaminated site remediation activities in Antarctica, as well as a valuable source of scientific information into the use of water treatment in cold regions.

On the 6th of October 2015, Veolia and the University of Melbourne signed a 5 year Memorandum of Understanding (MoU) to collaborate on research into better water and waste treatment technologies for both municipal and industrial sectors. The first collaborative project to be launched under the umbrella of the 5 year MoU focuses on the remediation of contaminated sites impacted by hydrocarbon (diesel) spills in Antarctica. Veolia supported the University of Melbourne on Antarctic Science Project 4029, through provision of remote technical and onsite operational support of the WTP at Casey Station.

Veolia provided technical design support, and an operator/maintainer was sent to provide operational support at Casey station, for Antarctic Science Project 4029. The aim was to repurpose and recommission the plant to treat diesel contaminated snow and leachate from sediments. Veolia staff worked with a team comprising staff from the Australian Antarctic Division and the University of Melbourne to commission, operate and monitor the water treatment process. Under the direction of University of Melbourne Project Team Leader, Kathryn Mumford, and AAD Project Officer Tim Spedding, Veolia staff worked with the project team members, who were able to successfully start up and operate the treatment plant, treating around 60,000L of contaminated water between the 22nd November to mid-December 2015.

Figure 2: ***Onsite at Casey Station, Antarctica***

The following sections describe the key activities undertaken with regard to pre-commissioning, commissioning and operation of the Casey WTP. In addition to this, options for possible future improvements to the process are discussed.

2.0 PRE-COMMISSIONING ACTIVITIES

The pre-commissioning phase of the Casey WTP project was conducted between the 19th and the 22nd of November 2015. Key activities included:

- Initial inspection and condition assessment of the plant. Condition was rated as good.
- Parts list for modifications compiled and parts sourced.
- Location for treatment identified and cleared ready for plant to be deployed.
- Plant brought onto site, further plant inspections to confirm planned modifications.

Modifications were determined to be raising the original ion exchange columns by 200mm. These columns were to be retrofitted to become activated carbon filters, including changes to plumbing, pipework and installation to allow for easier disconnect/reconnect.

Further pre-commissioning activities included:

- Electrical testing of mixers, pumps, lights, heaters and GPOs
- Repair of seized centrifugal pump and reconnection.
- Bund construction for storage of contaminated meltwater for treatment in WTP.
- Filter columns raised and re-plumbed.
- Check operations of chemical dosing system.

Figure 3: ***Raising the columns in the Casey WTP***

3.0 COMMISSIONING ACTIVITIES

The commissioning activities occurred from the 22nd of November 2015 and included:

- Hydrostatic testing of plant and repair of leaks
- Set up of feed and discharge lines.
- Open up, fill filter columns with granular activated carbon (GAC) and close up.
- Connect sump pump in bund and run WTP discharge line to permeable reactive barrier.
- Start-up plant, monitor and adjust flows across process.
- Monitor WTP ambient air quality for volatile hydrocarbon.

Figure 4: ***Connecting up the pump pipework prior to commissioning***

4.0 OPERATION

4.1 Plant Operation

The optimum plant flowrate was found to be 6 litres/min. Based on the experience of the Veolia and University of Melbourne operators onsite, this flow was found to produce sufficient contact time in the filter columns to remove hydrocarbon without it breaking through too quickly. Overall it was found that around 15,000L of hydrocarbon-contaminated snow-melt could be treated before the filter column media reached exhaustion.

The upstream processes (flocculation tanks, settler, bag filters) were all commissioned and operational from commencement of plant start-up. However in the first phase of the project, other than hydrostatic and electric testing, the chemical dosing system was not commissioned and operational. It was later in the season that the chemical dosing system was commissioned.

Operational activities were conducted between the 23rd November and the 10th December, at which point the Veolia operator formally handed over to the project team who were continuing for the remainder of the summer. Operational activities included:

- Plant condition and monitoring checks.
- Repair equipment if needed and rectify any plant issues (ie frozen pipework, split valves etc.)
- Start-up plant and monitor flows on an hourly basis.
- Sample collection.
- Air quality monitoring
- Discharge water quality monitoring
- Bag filter and filter column pressures and flows.
- Change out of GAC as required.

Further activities included procedure writing and training of new operators.

4.2 Filter Column Media

The initial composition of the filter columns was a mixture of zeolite and GAC. This was found to reach adsorption capacity too quickly and hydrocarbon started breaking through. Hence it was decided that the columns would be filled with 100% GAC. The volume of carbon used in each column was determined to be around 60L.

4.3 Bag Filters

The plant uses 2 x bag filters upstream of the filter columns. These are used to strain out coarse particles that potentially carry over from the settler due to the extremely low water temperatures. This is to prevent blockage of the GAC filter columns. The 5 micron filter bags tended to foul up quickly and cause a drop in flow across the process. This is likely due to no upstream chemical dosing in the early part of the season to encourage coagulation/flocculation and settling of particles. It may also be that the 5 micron bags are too fine and a coarser filter material (i.e. 10 micron) may be needed.

5.0 DISCUSSION

5.1 Operational Performance

The GAC filter columns in the Casey water treatment plant have the following design characteristics:

Table 1: ***GAC filter column design characteristics***

Column height	1.5	m
Column dia.	0.25	m
Columns in train	2	
trains in parallel	2	
Volume GAC	0.06	m3
Height GAC	1.2	m
Volume/train	0.12	m3
Total vol.	0.24	m3
Plant flowrate	6	L/min
Train flowrate	3	L/min
EBCT	40	min

A short review of literature of activated carbon adsorption of hydrocarbons, indicated recommended empty bed contact times (EBCT) of up to 120 minutes and a range of typical adsorption capacities.

Table 2: ***EBCT and adsorption capacities for GAC from literature***

Parameter	Contact	Concentration-range	Max Adsorption Capacity
	time (min)	mg/L	mg/g GAC
*Toluene	?	1.6-4.4	8.75
**Naphthalene	120	0-5	10
**Naphthalene	40	0-5	6

*Chemistry of Water Treatment, Second Edition Samuel D. Faust, Osman M. Aly July 1, 1998 by CRC Press

**C. Valderrama a, J.L. Cortina b,*, A. Farran b, X. Gamisans a, C. Laoa, Kinetics of sorption of polyaromatic hydrocarbons onto granular activated carbon and Macronet hyper-cross-linked polymers (MN200), Journal of Colloid and Interface Science 310 (2007) 35–46

5.2 Recommendations

Based upon operational observations between November 22nd and December 10th 2015, as well as an analysis of operational and design data from the Casey WTP, the following options are put forward for consideration for further investigation:

Option 1:

Optimise the removal of hydrocarbons through the GAC filter columns using existing equipment with no modifications. This would involve adjusting flows to achieve a better EBCT, commission chemical dosing to achieve to an optimum pH range and sourcing the most suitable GAC for hydrocarbon adsorption capacity. This approach would have limitations. The flows would need to be reduced down to 2 L/min to achieve a more suitable EBCT, posing issues with freezing of lines. However, this would be the cheapest and simplest option.

Option 2:

Replace the existing GAC filter columns with larger columns. This would be to achieve an EBCT of at least 120 minutes and provide greater overall adsorption capacity to reduce change out frequency. This would be the most expensive option in terms of capital cost.

6.0 CONCLUSION

A mobile water treatment plant (WTP) was constructed in 2002 for deployment to remote Antarctic contaminated sites for use during clean-up activities. In 2015 a project was launched to repurpose and recommission the above-mentioned mobile water treatment plant to treat diesel contaminated snow and leachate from sediments. Veolia technical and operational staff were involved in this project under the project management and direction of the Australian Antarctic Division and the University of Melbourne.

Modifications were made and the plant was recommissioned in November 2015. The optimum plant flowrate was found to be 6 litres/min. Based on the experience of the operators onsite, this flow was found to produce sufficient contact time in the retrofitted activated carbon filter columns to remove hydrocarbon without it breaking through too quickly. Overall it was found that around 15,000L of hydrocarbon-contaminated snow-melt could be treated before the filter column media reached exhaustion.

Possible future improvements to the plant and treatment performance is to either increase GAC filter size, or slow down plant flow. Either of these could be used to improve the empty bed contact time and hydrocarbon removal from the contaminated water.

7.0 ACKNOWLEDGEMENTS

The author would like to acknowledge project partner, the University of Melbourne, as well as the Australian Antarctic Division and funding via the Antarctic Science Program (Project 4029).

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FROM Mi WATER TO Mi SEWER
USING IoT TECHNOLOGY TO MONITOR SEWER NETWORKS

Paper Presented by:

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42nd Annual WIOA
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FROM Mi WATER TO Mi SEWER
USING IoT TECHNOLOGY TO MONITOR SEWER NETWORKS

Matthew Harris, *Plumber,* Mackay Regional Council

ABSTRACT

Obtaining a better understanding of the manner in which our sewer networks perform under differing conditions has been a long standing objective for Mackay Regional Council's (MRC) water & sewerage team.

After exploring a range of approaches and technology options, MRC focussed on partnering with a Sydney based Technology Company to develop a relatively low cost monitoring device that could be used in conjunction with an existing LPWAN communications network that MRC has already invested in for the use of Mi Water

Although the project is still in the early trial stage, the results are very encouraging and we believe it can play a significant role in helping us better understand and manage our sewer network, definitely from an ongoing operational perspective, but also from a longer term planning and decision making perspective

1.0 INTRODUCTION

Gaining a better understanding of the way our sewer networks perform under differing weather conditions has been a long term objective for Mackay Regional Council's (MRC) water & sewerage team. Primary areas of concern and focus were:

- Understanding how the sewer responded in the different catchment areas to wet weather events. Mackay's sewer areas are relatively flat, and can see the water table respond rapidly. It is always a challenge to figure out how, when and where to deploy crews during rainfall events.
- Council has invested significant dollars in I&I inspections over the years, but these investments have not seen an evident reduction in flows or lower overflow events in wet weather.
- Response to dry weather events are always reactive. Council's goal is to be able to move into a more pro-active mode.

Due to the relatively flat landscape of Mackay, our sewer network has a high ratio of sewer pump stations compared to kilometres of mains. All of these pumps stations are on the SCADA network, which provides a certain level of information on wet well levels, pump run times etc. but for better operational purposes, it was determined that more granular information would be desirable, provided that such information can be captured at a reasonable cost.

2.0 DISCUSSION

2.1 Defining the Deliverables

Prior to embarking on a search for a technical solution that could deliver the required data at a cost point that would enable the establishment of a positive business case, several internal discussions were initiated to understand for ourselves and thereafter clearly document the exact nature of the data required.

In addition to defining what was needed, these discussions also focussed on identifying what was not needed.

Previous experience with other technology projects has indicated to us that in order to develop a solution that best delivers value for money, defining what was not needed was an important element. Technical products are generally designed to address a range of requirements. This can mean that a product can have several features that are not needed. While this is not an issue in itself, it does mean that more likely than not, we will be paying for features that we will never need or use.

The discussions identified the following data requirements

- Sewer Levels at selected locations (in manholes)
- Status of overflow outlets
- Rainfall at a catchment level (BoM has only 2 locations for Mackay)

Drilling down into these requirements based on the desired final outcomes, it was noted that:

- Exact depth of the water in a sewer manhole was not essential, but indications at 2 levels pipe full, manhole full would be all the information required
- For overflow outlets, the critical information was whether or not the outlet was discharging.
- Detailed information of the millimetres received was needed.

Several other requirements such as the need for this information in near real time (mainly for operational purposes), and the ability of data to be fed into a general data warehouse, were also identified.

2.2 Search for a Solution

A scan of the market was undertaken to assess available solutions. While there were products available, the cost per installation tended to be quite high, primarily driven by a high level of features which were in excess of our requirements. It became evident early in the process that the whole of life costs for these initial devices would not enable the establishment of a positive business case.

At this time, MRC’s Water & Waste business had already set up a dedicated low cost communications network using LPWAN technology for the automated monitoring of water meters. The communications services supplier, who had also provided the automated meter reading (AMR) devices, indicated that they could develop a very basic sewer monitoring device that could potentially be delivered at a fraction of the then existing market prices of the other devices.

As we had done with the Smart metering project for water, we worked with the manufacturer to design/develop a low cost device that delivered the basic requirements. The first iteration of the device consisted of two float switches, one positioned at the pipe full level and the other just below the overflow level, attached to the same transmitter used for the water meter AMR devices. The intent was to make a transmission as soon as the float floated up beyond the horizontal level. A single float switch version was used to monitor the release from overflow outlets.

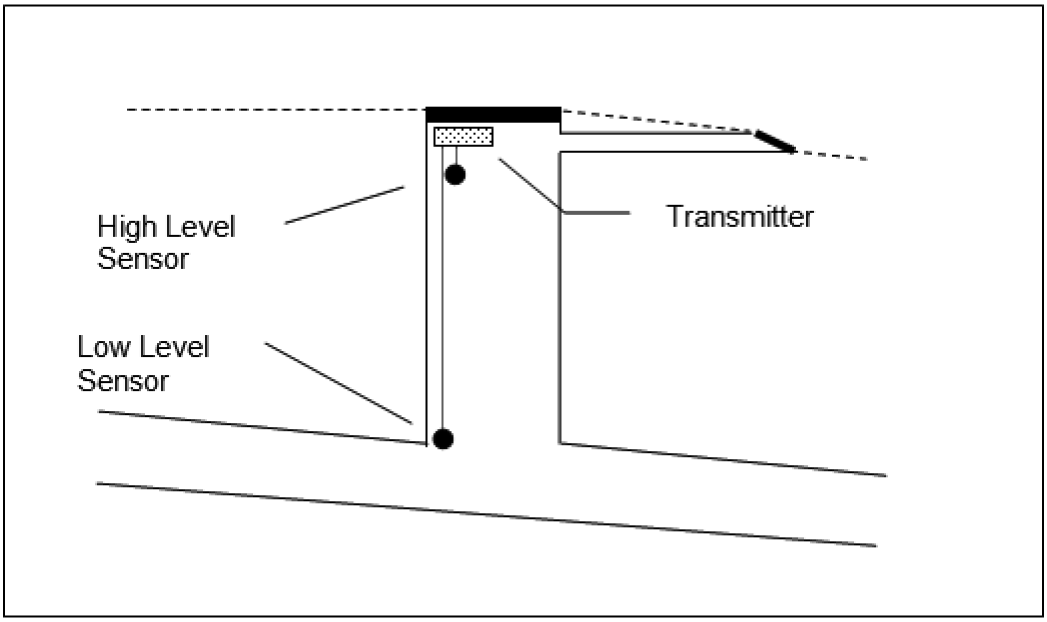

Figure 1: ***Diagrammatic Illustration of Concept***

For monitoring of rainfall, a gauge that could capture data at 5 mm intervals was attached to the transmitter.

2.3 Initial Trial

As an initial trial, seven sewer taggle devices were installed at selected locations. This was later extended to around 50 devices.

At the time of embarking on the trial, it was estimated that a commercial version of the prototype could be delivered at a cost point of around $250 - $300 (device cost only, excluding cost of installation). Such a cost point would enable the roll out of adequate devices to cover the entire sewer network at a fairly detailed level.

MRC has around 10,000 sewer manholes, and it was estimated that installing monitoring devices in 8-10% of these manholes would provide a good view of the manner in which the network was performing.

The expectation was that the battery within the transmitters would last around 5 years. With an ongoing data transmission cost of around $10 per device per year, operational costs would also be within manageable limits, provided the devices did not require any extensive maintenance over its life.

2.4 Monitoring System

In order to make practical use of the data captured by the monitoring devices, some basic software and alerting system was required. As a first instance, the device supplier also supplied a web based interface that enabled the visualisation of the status of each manhole being monitored.

Suburb	Address	Tag	Bottom sensor	Top sensor	Last updated
Mobile	Survey Tag	303873			
HOGANS POCKET	HP Detention Res High	57363	LOW	NO SENSOR	10:01 28-Apr
HOGANS POCKET	HP Detention Res Low	57356	LOW	NO SENSOR	10:12 28-Apr
HOGANS POCKET	HP Detention Res Overflow	57350	LOW	NO SENSOR	09:50 28-Apr
EIMEO	EI01/01/04A BUSSUTTIN DRIVE	11527	LOW	NO SENSOR	09:44 28-Apr
EIMEO	EI01/01/14 MATTERSON AVE	11500			
MACKAY	PSBE04 AVOCADO COURT SPS	11519	LOW	LOW	09:50 28-Apr
MACKAY	PSNM01 BASSET STREET SPS	11529	LOW	LOW	10:15 28-Apr
MACKAY	PSNM01 BASSET STREET SPS	11538			
MACKAY	BEACONSFIELD SPS1	11508	LOW	LOW	09:19 28-Apr
MACKAY	BEACONSFIELD SPS1	11515	LOW	LOW	09:58 28-Apr
MACKAY	PSBE01 BEACONSFIELD NO 1 SPS	11510			
MACKAY	PSBE01 BEACONSFIELD NO 1 SPS	11511	LOW	LOW	09:29 28-Apr
MACKAY	BEDFORD ROAD	11439			
MACKAY	PSSA01 BILTOFT STREET SPS	11520	LOW	LOW	09:36 28-Apr
MACKAY	PSSA01 BILTOFT STREET SPS	11524	LOW	LOW	09:50 28-Apr
MACKAY	PSSA01 BILTOFT STREET SPS	11537	LOW	LOW	08:01 28-Apr
MACKAY	1 BLACKWOOD STREET	11386	LOW	LOW	07:37 28-Apr
MACKAY	PSWA01 BOLD STREET SPS	11506			
MACKAY	CELEBER DRIVE	5420			
MACKAY	PSAN01 COLES ROAD	11541			
MACKAY	PSSM22 CONNORS ROAD SPS	11536			
MACKAY	PSMC21 CULLEN STREET SPS	5409			
MACKAY	DANIEL STREET	5525	LOW	LOW	09:51 28-Apr
MACKAY	DAVID MUIR STREET	11398			
MACKAY	9 EDWARD STREET	11408			
MACKAY	PSNM02 FORGAN STREET SPS	11521	LOW	LOW	09:39 28-Apr
MACKAY	PSNM03 GOOSEPONDS SPS	11504	LOW	LOW	10:48 28-Apr
MACKAY	PSNM03 GOOSEPONDS SPS	11526	LOW	HIGH	10:25 28-Apr
MACKAY	GORDON ST SPS	11345			
MACKAY	PSMC12 GRAFFUNDER STREET SPS	11533	LOW	LOW	09:38 28-Apr

<u>Figure 2:</u> ***Initial Data Visualisation Site***

The system also has the capability to send SMS alerts to an identified mobile number on the activation of any one of the sensors deployed.

2.5 Practical Application

This initial trial helped establish the usefulness of the information very early on when it assisted in identifying and preventing several dry weather chokes in the sewer system.

The sewer networks crew soon became dependent on the site to obtain information on the network performance in the areas where the sensors had been deployed.

Around 18 months into the trial, we began to notice device failures. Investigations by the manufacturer indicated the failure were due to the battery life running out well before the anticipated time frame. Further investigations by the manufacturer identified two possible causes for the early failures.

- It was noticed that a sizeable portion of the battery was already drained at the point of installation. This was thought to be caused by transmissions forced due to vibrations during transportation from point of manufacture to installation site. Based on the then design of the device, the float switches were already attached and the device activated at the point of manufacture.
- Some devices also registered very frequent movement between on and off positions, which again forced an exceptionally high number of transmissions within a very short period of time. This was thought to be caused by water turbulence just as the pipe full level was reached, where the float switched bobbed up and down due to the turbulence.

Due to these unforeseen circumstances, a redesign of the device was required to ensure that the original operational life of the battery was achieved. The manufacturer went back to the drawing board to come up with a revised version of the device that was capable of dealing with the identified operating conditions.

2.6 Data Management System Development

It was recognised from the outset that in order to make full use of the data generated through the sensors, a well-designed and robust data management system would be essential. As such, MRC also embarked on a parallel project to design such a system.

MRC had already done a lot of the work in successfully designing a meter data management system (MiWater) to manage the large amount of data being generated through the AMR devices. Much of the back end development done for MiWater could be used for the sewer management system as well (named MiSewer), with the bulk of the work focussed on the front end visualisation.

2.7 Trial - Phase 2

The manufacturer's solution to addressing the issues faced by the current design of the device was to use a more advanced version of the processor (i.e. embed processing capability within the device located in the manhole). This enabled the coding of logic into the device to prevent unnecessary transmissions. The devices were also redesigned so that the float switches were now connected to the transmitter in a detachable manner.

A more advanced processor within the transmitter as well as incorporating high quality connectors that can withstand the harsh environment within a manhole meant that the previously estimated cost per device changed significantly. However at around $500 per device it was still at a very steep discount to other alternatives.

At the moment MRC has deployed just six of the new devices with another 80 expected to be delivered towards the end of May.

The initial phase of the MiSewer data management system is also now in operation with further refinements due in June this year.

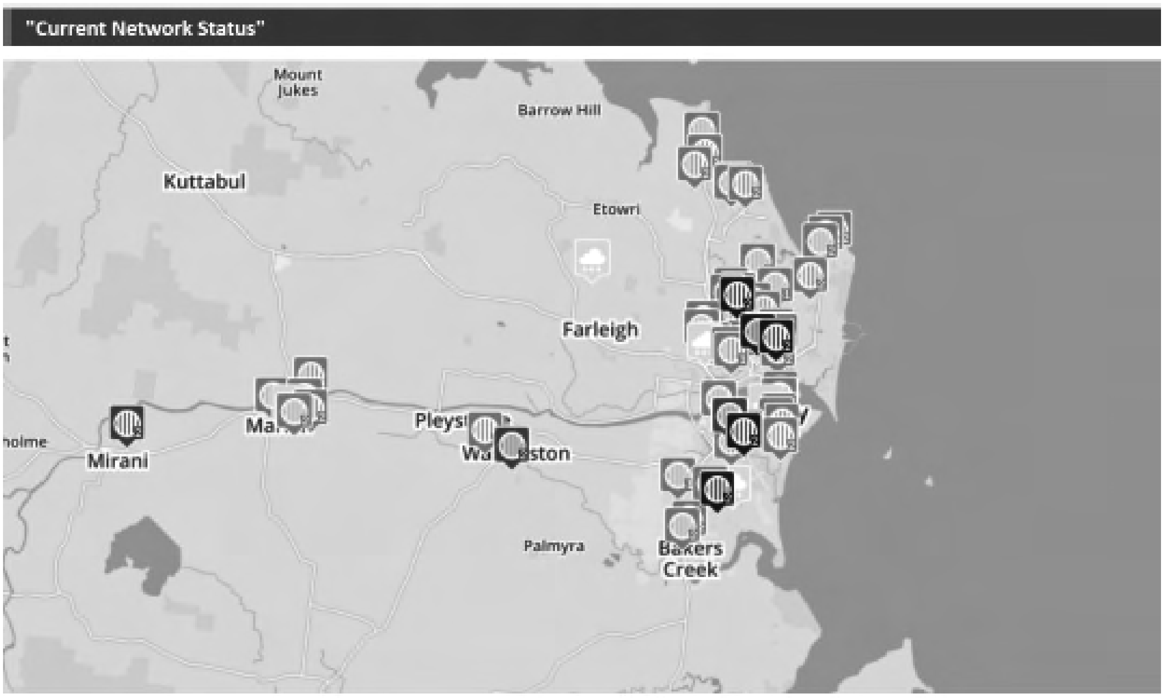

Figure 3: ***Initial Data Visualisation in MiSewer***

3.0 CONCLUSION

As indicated previously, the operational value of the sensors and the alerts generated therefrom has been established through the first phase of the trial. These alerts are very valuable in dry weather situations and has helped move the response from a reactive to a proactive mode. In wet weather situations, getting feedback from where the impact is being felt most helps in deciding on crew deployments

While more work needs to be done in the data analytics space, there is definite value in the insights gained through the data on the relationship between duration and intensity of rain and the flows (and overflows) experienced within the sewer network. Such insight can help direct investments such as I&I inspections (e.g. smoke testing) and sewer relining to areas with the highest potential for return on investments.

However, a crucial element yet to be established is whether the devices are capable of delivering the data at a whole of life cost that enables the establishment of a positive business case. While the estimated cost of the devices are within acceptable ranges, unless the device can achieve its full designed life in the field, the whole of life cost will become prohibitive. Phase 2 of the trial is expected to establish that aspect of the product viability.

4.0 ACKNOWLEDGEMENTS

The contributions to the project by the following suppliers/contractors are acknowledged:

- Taggle Systems - Device and Communications Services
- Tyeware - Systems Development
- Stephen Fernando MRC

The efforts by Mackay Regional Councils Sewer Network crew installing, monitoring and testing the devices and the data systems is also acknowledged.

CAPTURE RATE & HYDRAULIC COMPARISON OF PERFORATED PLATE VERSUS HONEYCOMB BAND SCREEN PANELS

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Rui Pu Yang, *Process Engineer,* Unitywater

ABSTRACT

Unitywater conducted a trial in December 2016 designed to measure and define the mass of dry screenings captured per ML of treated wastewater at an Australian Sewage Treatment Plant (STP). The trial would be conducted at the inlet works of Maroochydore STP to determine the weight of screenings captured pre-screen and post screen. It is widely accepted that using an inlet works screen with a high Screen Capture Rate (SCR) and increasing removal of inorganic material reduces stress on downstream mechanical and biological processes. Unitywater wanted to conduct SCR trials on band screens, as they are widely accepted as having the highest SCR on market. The focus of the trials would be to quantify the capture rate of two band screen panel types at the Maroochydore STP and to quantify the dry kilograms of screenings per ML of wastewater. The technologies tested were a perforated plate Centre-Flo band screen and a new honeycomb panel Centre-Flo band screen. The data would also be compared to a third hooks and links style Centre-Flo band screen, also installed at the site.

1.0 INTRODUCTION - BACKGROUND

In 2009, Mark Wilson and Michael Thomas co-authored a paper (Wilson 2009) that focused on identifying the best screening technologies available to the market in Australia. At the time the paper cited only one notable study available to quantify Screen Capture Rates (SCR's) for inlet screens at wastewater treatment plants which was carried out by UK Water Industry Research between April 1998 and March 2001. A subsequent study was completed at UK National Screens Evaluation Facility, Chester-Le-Street STW, between March 1999 and December 2015.

Wilson and Thomas wanted to test and identify the most efficient screen type for different applications. Their motivation to write the paper was the lack of information available specifically in Australian conditions. To conduct this study a cooperative effort was required by the South East Queensland local utilities to evaluate inlet screen performance based on classifying screens into six screen type categories. The paper concluded that of the 31 wastewater treatment plants observed, no evidence supported a clearly superior screen type. The Australian paper acknowledged that band screens exhibited the highest specific capture rate in the UK trials in the range of 80-90%.

In 2014, Michael Thomas from Unitywater attended a WSAA Technology Approval Group (TAG) Forum for innovative new technologies. Aqseptence Group's Australian designed and made Johnsons Screens perforated plate Centre-Flo band screen was being presented as an evolving technology. Michael Thomas wished to conduct trials with the perforated plate band screen compared to older hooks and links (also known as lace link) band screens. In 2015, a capacity upgrade at the Maroochydore STP provided the opportunity to install a Johnsons Screens perforated plate Centre-Flo band screen. That allowed the SCR trial to be conducted. Aqseptence Group located in Brisbane, were the ideal partner for the SCR due to their proximity to Maroochydore and proactive development of inlet works technology.

2.0 TRIALS

2.1 Trial 1: Perforated Plate Panels

In December 2015, Aqseptence Group's perforated plate Centre-Flo band screen was installed beside two hooks and links band screens at Maroochydore STP to upgrade the plant capacity. The inlet works building was originally designed with 4 identical inlet channels which simplified the installation of a third screen and allowed for side by side comparison. The hooks and links band screens are very operator intensive and are not a favoured technology due to the time required to manually clean inorganic material from the stainless steel hooks and links.

Once the perforated plate band screen was installed, the operators immediately noticed the screening bins required emptying daily, rather than every second day. The operators also noted that the perforated plate band screens required far less operator input or cleaning. The operators needed to manually clean the hooks and links band screens daily while the new screen only needed to be inspected weekly, with minimal cleaning required. A significant increase in screenings removal was noticed immediately, however both types of screens were being operated simultaneously and discharging into a common bin. Therefore a focused SCR trial would be required to identify an accurate SCR % of the perforated plate screen.

Aqseptence Group, who manufactured both the hooks and links band screens and the perforated plate band screens, conducted the trials at Maroochydore STP. Detailed planning was pivotal for the success of the SCR trials as the site would need to remain in continuous operation. In addition, a risk assessment of the trial methodology was conducted by Aqseptence Group and Unitywater which resulted in the requirement for a suitably qualified team to manage confined space and working at heights risks.

The SCR trials were conducted by the team in December 2016. The methodology used to conduct the trials replicated the methodology used by UK Water Industry Research to conduct trials between 1998 and 2001. The trial calculated the screen capture rate based on mass balance between the upstream captured mass and downstream captured mass. To capture the upstream and downstream mass, 2mm Copasacs were mounted onto an open fibreglass nose box which was secured onto a long SS frame. The Copasacs was completely submerged into the wastewater channel for a duration of 3 minutes. After each 3 minute trial, the Copasacs was removed from the nose box and allowed to dry whilst the post screen trial was conducted immediately to alleviate any loading fluctuations. Each SCR sample was collected during morning peak flow conditions each Copasac was measured to the accuracy of 1 gram by a set of digital scales.

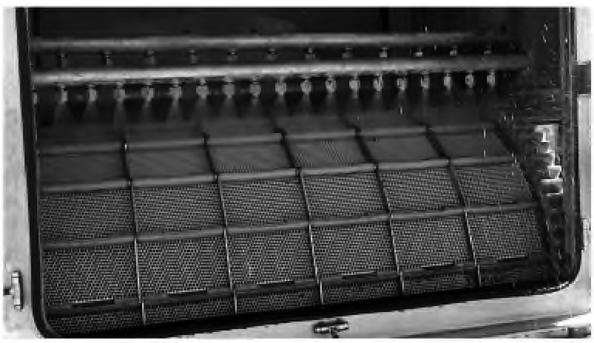
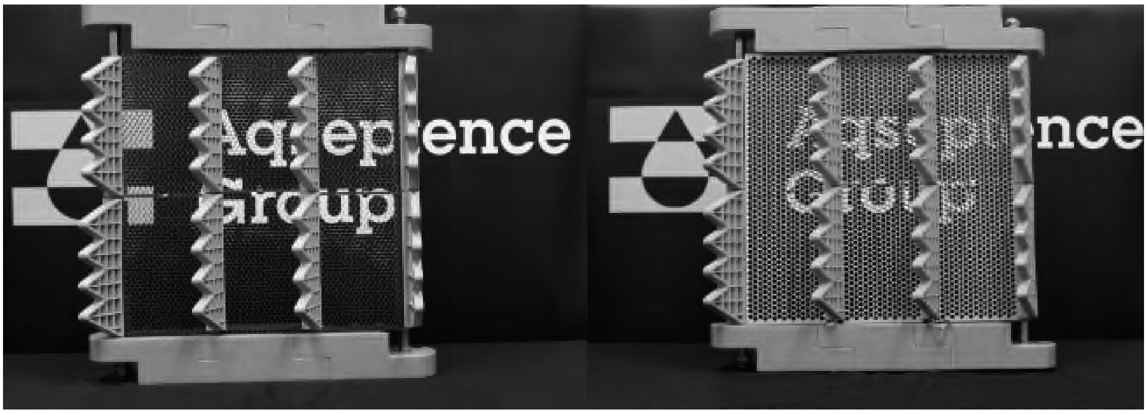


Figure 1: ***Perforated plate band screen during wash cycle with a single wash water sparge operating***

2.2 Trial 2: Honeycomb Panel

The purpose of this trial was to quantify the hydraulic capacity and SCR increase of a new band screen honeycomb panel that was developed and patented in 2016. Hydraulic modelling showed up to 40% increase in hydraulic capacity due to an increase in panel open area from 65% (5mm aperture on perforated plate panel) to 92% (nominal 5mm aperture on honeycomb panel). The honeycomb panel is Stainless Steel 316 while conventional plastic perforated plate panels are usually made of polypropylene or ultra-high molecular weight polyethylene. The open area improvement can be visually observed in the following photo comparison of the new



<u>**Figure 2:**</u> ***Vertical Comparison between Honeycomb Panel (left) and Perforated Plate (right)***

In December 2016, Unitywater was offered the opportunity to participate in a world-first trial using the new honeycomb panel technology by means of retrofitting into their current perforated plate band screen. This would allow a direct comparison of the SCR and hydraulic performance between the two panel types under the same plant conditions.

In order to verify the predicted increase in hydraulic capacity, the time taken to reach the upstream maximum level under known instantaneous flow rates was conducted for both the perforated plate band screen and honeycomb panel band screen. During this “fill time” measurement, the band screen was not cycling (kept in stationary mode) which allowed the screen to blind and therefore increase in upstream level. It was anticipated that a unit with increased hydraulic capacity would have a relatively proportional increase in “fill time”.

The SCR trial methodology for testing the honeycomb panel was identical to that outlined in Trial 1.

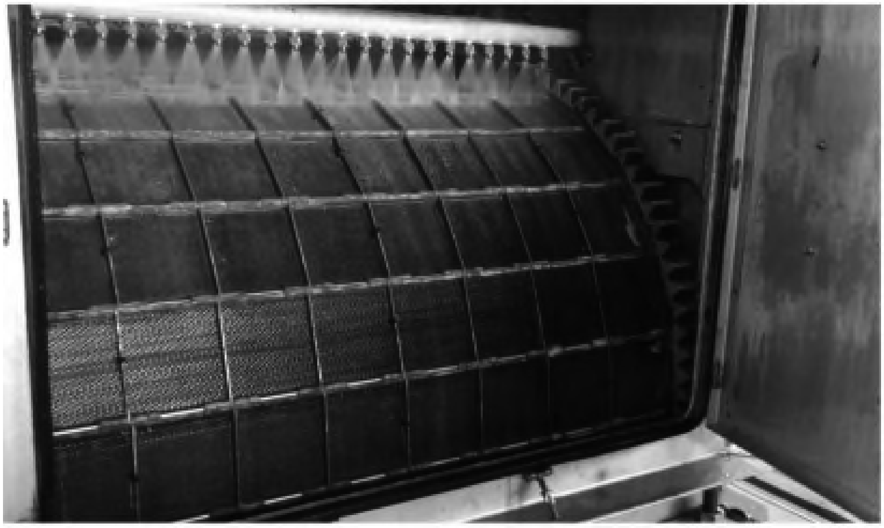


Figure 3*:* ***Honeycomb panel band screen during wash cycle with a single wash water sparge operating***

3.0 ANALYSING RESULTS FROM THE SCR TRIALS

When the data set was analysed, the Johnsons Screens Perforated Plate Band Screen achieved a capture rate of 85%, and the Johnsons Screens Honeycomb panel Band Screen achieved 89%. Each SCR sample was taken over a period of 3 minutes and resulted in a wet weight of captured screenings measurement. The instantaneous flow rate was also measured for the duration of the SCR trial and it was averaged over those same 3 minutes for the trial period. The weight of each sample was then converted to wet kg per second by removing the weight of the Copasac then dividing by the sample time of 3 minutes.

Dry screenings kg/ML was then calculated by assuming 5% dry solids in each sample and upscaling the sample weight by the cross-sectional area of the Copasac by the cross-sectional area of the channel (upstream channel depth was measured at time of sampling). This method allowed the relative comparisons to be made across all upstream and downstream samples. This method calculated the cross sectional % based on upstream water depth only and did not account for the through screen head loss. This method could artificially inflate the downstream capture rate due the increased flow during the sample period however, as same method was used for both sets of screens, the same error would have occurred to both data sets. In calculating the cross-section it was assumed that inorganic material is uniformly distributed in the cross section of the channel sampled which may not be the case during lower flows or when less turbulence is present. The SCR% was then calculated based on the difference in dry kg/ML between the upstream and downstream samples. A total of twenty samples were taken for both the perforated panel band screen trial as well as the honeycomb band screen trial however due to fouling or tearing of the Copasac during the sample only 6 and 7 valid samples for perforated and Honeycomb panel respectively.

Separate analysis of bin collection data indicated a 40% increase in screened solids were removed from site when the honeycomb panel band screen was in use compared to the perforated plate band screen. This separate analysis supports operator feedback of an increase in bin rotation from every 24 hours to every 17 hours, which also supports the claim of an increased SCR with the honeycomb panel.

During the SCR trial period the following average weights of screenings removed by the screens at the Maroochydore STP was calculated:

$$SCR\% = \frac{\text{Average Dry}\frac{\text{KG}}{\text{ML}}\text{upstream} - \text{Average Dry}\frac{\text{KG}}{\text{ML}}\text{downstream}}{\text{Average Dry}\frac{\text{KG}}{\text{ML}}\ upstream}$$

Up until 2015 - Hooks and Links Band Screens removed 365T of screenings per year.

2015-2016 – Perforated Plate Band Screen removed 730T of screenings per year.

2017-future – Honeycomb Panel Band Screen removed 1030T of screenings per year.

4.0 ANALYSING RESULTS FROM THE HYDRAULIC CAPACITY TRIALS

The results of these hydraulic trials showed the new honeycomb panel with higher open area provided a 4% increased SCR while also providing a 29% increase in hydraulic capacity over the perforated plate band screen. This hydraulic capacity increase was calculated based on a total of twenty samples where the fill time was measured and compared to the average instantaneous flow rate during that fill time. All sample results were normalised to a flow rate of 1000 L/s to allow for a relative comparison.

When the trials started at an upstream level of 600 mm the average fill time was 287 seconds for the perforated plate band screen and 364 seconds for the honeycomb panel band screen. When starting from an upstream level of 500 mm, the average fill time was 226 seconds for the perforated plate band screen and 296 seconds for the honeycomb panel band screen. A trial based on headloss differential would have been a more conventional approach; however this was not possible as the inlet works channel profiles are sloped both upstream and downstream of the band screen.

$$Hydraulic\ capacity\ increase = \frac{\text{Normalised fill time of perforated plate band screen}}{\text{Normalised fill time of honeycomb panel band screen}}$$

Figure 4*:* ***Band Screen 3 at Maroochydore STP where trials were performed. Image above is with complete band constructed from honeycomb panel***

5.0 CONCLUSION

Unitywater has been able to validate the claimed screen capture rate reported in the UK National Screens Evaluation Facility in Australian conditions. Through field trials it could be substantiated that Aqseptence Group's Perforated Plate Centre-Flo band screens have an average capture rate of 85%. Aqseptence Group's new Honeycomb Panel Centre-Flo band screen achieves a higher SCR at 89% with a 29% increase in hydraulic capacity. Unitywater can now quantify that on average, 12 kg of dry screenings are removed per ML of wastewater by a perforated plate band screen and 17.5 kg of dry screenings is removed per ML of wastewater by the new honeycomb panel band screen.

Bin collection data and operator feedback showed a 40% increase in solids removed off site when comparing the perforated plate to honeycomb panel band screen. This supports the conclusion of an improved SCR performance of the honeycomb panel band screen.

The results of these trials have far reaching implications for the Australian water industry and the water industry globally. The honeycomb band screen can provide brownfield inlet works with higher hydraulic capacity while also providing greater than 85% removal of the screenings material, preventing downstream equipment and process issues. New treatment plant design will also benefit from the honeycomb panel by offering smaller size inlet works structures and reducing delivery project costs.

6.0 ACKNOWLEDGEMENTS

Special thanks to Michael Thomas of Unitywater and Al Law, lead operator at the Unitywater Maroochydore STP, for providing access and working through the trials with us over the last 18 months. Also to Chris Susanto from Aqseptence Group for his support throughout the trials.

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